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Concrete Pavement Test Projects In Connecticut and Indiana

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Concrete Pavement Test Projects In Connecticut and Indiana

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Contents

Performance of an Experimental Project to Determine the Efficiency of Several Plain And Reinforced Concrete Pavements

CARL E. VOGELGESANG, Chief Engineer, and
WILMER E. TESKE, Research Engineer
State Highway Department of Indiana

This report describes the design and performance of an experimental concrete pavement constructed in 1931 a few miles north of Indianapolis on State Road 29. The purpose of this experiment was to determine the relative efficiency of several designs of reinforced concrete pavement, expansion joints, and contraction or dummy joints. Nine special design sections, each approximately 2,700 feet in length, and seven standard sections were incorporated in 8.35 miles of test pavement. Special design features in the nine sections included variations in the type and amount of reinforcement, type and spacing of transverse joints, and type of joint filler.

Numerous observations and at least two major surveys had been made of the test project prior to 1954. This report includes information from earlier reports, along with information obtained in 1954, after 23 years of service. Included in the 1954 observations were (a) the location of all transverse cracks, (b) measurement of faulting at all joints and transverse cracks, (c) pavement condition in the vicinity of joints and cracks, (d) general soil types encountered beneath the pavement, and (e) surface roughness indices for the various sections as measured by the Bureau of Public Roads' Road Roughness Indicator. Traffic data were furnished by the Highway Planning Survey of the State Highway Department of Indiana.

Consistent with other test projects, the more heavily reinforced sections developed closer transverse crack patterns than did the lesser reinforced sections. After 23 years of service only minor damage has resulted from pumping action of the slabs. Special consideration is given in this report to the cracks that indicate a structural failure either by faulting, spalling, or ravelling, and which have required some maintenance. In general, wire mesh reinforcement was found to be more effective in preserving the continuity of the pavement than the bar mat type of reinforcement. Based on surface roughness measurements made in 1941 and 1954 some reinforced sections have improved in surface smoothness during the past 13 years, while all standard sections have shown a decided increase in roughness.

● IN 1931, the state of Indiana designed and constructed its first reinforced concrete pavement as an experimental project in order to determine the relative efficiency of several designs of plain and reinforced concrete pavements and of expansion and contraction joints. This is a report of the 23 year performance of the project located just north of Indianapolis on a section of SR 29. This portion of the old Michigan Road had been taken into the state system in 1923 and maintained as a gravel road until 1931.

As this is the first published report on this project, it will be necessary to acquaint the reader with some of the general conditions and prevailing practices under which this project was constructed.

The site selected for the construction of the experimental project was the section from the Marion-Hamilton county line north 8. 2 miles to the junction with SR 32. The centerline of the new road approximately coincides with that of the old gravel road, but variations exist between their respective grade lines. The terrain throughout the area is gently undulating glacial till dissected by several small creeks and streams. In general, surface drainage is good throughout the test project. It was believed, however, that the existance of the gravel road along the line of the new road had resulted in a considerable variation in the quality of the subgrade immediately beneath the test

TABLE 1

SUMMARY OF SOIL SAMPLES TAKEN TO A DEPTH OF SIX INCHES BENEATH THE OUTSIDE BOTTOM EDGES OF THE PAVEMENT

"A" Layer		"B" Layer		"C" Layer	
New Grade Below That of Old Gravel Road					
11[a]-SaL	A-2-4(0)[b]	1-SaCL	A-4(4)	1-Sa(ORM)	A-1-b
		1-SaCL	A-6(6)		
1-Sa(ORM)	A-1-b	4-CL	A-6(8)	1-L	A-6(5)
		4-CL	A-4(4)		
1-L	A-6(5)	1-SaL	A-4(4)		
		1-Sa(ORM)	A-1-b		
New Grade Common to That of Old Gravel Road					
6-SaL	A-2-4(0)	1-Sa(ORM)	A-1-b	2-Sa(ORM)	A-1-b
		1-Sa	A-2-4(0)		
1-Sa	A-2-4(0)	1-SaL	A-4(1)	2-L	A-4(4)
		1-SaL	A-4(2)		
		1-SaL	A-4(4)	1-CL	A-6(8)
		1-CL	A-4(4)		
		1-CL	A-6(8)		
New Grade Above That of Old Gravel Road					
5-SaL	A-2-4(0)	1-Sa(ORM)	A-1-b	1-Sa(ORM)	A-1-b
		1-SaL	A-2-4(0)		
1-Sa	A-2-4(0)	1-SaL	A-4(1)	1-CL	A-6(8)
		1-SaL	A-4(2)		
1-Sa(ORM)	A-2-4(0)	1-SaCL	A-4(4)		
		1-SaCL	A-6(6)		
		1-CL	A-4(4)		

Abbreviations Used in Soil Texture Classification

Sa - Sand	L - Loam	SaCL - Sandy clay loam
SaL - Sandy loam	CL - Clay loam	ORM - Old road metal

[a] Numbers indicate the number of locations at which the material was encountered.
[b] U S Bureau of Public Roads Soil Classification

pavement. Earlier performance surveys report the development of localized areas where pavement surface distress was more than normal. Although only a minimum of grading was done during the new construction, some cuts and fills up to four and eight feet, respectively, did develop. As there was no accurate record available of the actual soils underlying the test pavement, a special soils survey was conducted as a part of the 23 year performance survey. The purpose of this survey was to obtain samples of the subgrade from immediately beneath the pavement edges. In all, some 40 samples of the upper six inches of the subgrade were obtained from 28 test pits located at random along the pavement edges. These test pits were located alternately on either side of the pavement at approximately 1,500-ft intervals. The exact locations were selected so that a variety of cut, fill and transition areas was represented.

The soil samples were taken to the laboratory where they were first arranged into 16 groups based on visual inspection. The soils in each group were then blended together and a representative sample taken for complete analysis. A summary of the classifications of these soils is given in Table 1. In general, the soils encountered immediately beneath the pavement were of a sandy texture and had a relatively low plasticity index. In only one instance was a soil heavier than an A-2-4(0) encountered adjacent to the underside of the pavement, and that was an A-6(5) loam which existed in a cut section where tar paper had been placed on the subgrade. With the exception of the old road metal, the soils ranged between an A-2-4(0) and an A-6(8) with the majority falling in the A-4 classification. Since these samples were taken in various types of sections ranging from cuts to fills, it is believed that they represent the general soil types existing immediately beneath the pavement throughout the test project except, perhaps, for some short localized areas.

Besides the variation in the type of soil underlying the test pavement, a considerable variation in soil conditions also existed. No special effort had been made to correct variations in moisture and load carrying capacity of the soils during construction. Because of these variations localized areas exist throughout the test project where more or less than average pavement distress had developed over the past 23 years. However, in view of the length of each test section (approximately 2,700 ft) it is believed that such variations exist within the limits of each section and were, therefore, not given special treatment in the analysis of the data presented in this report.

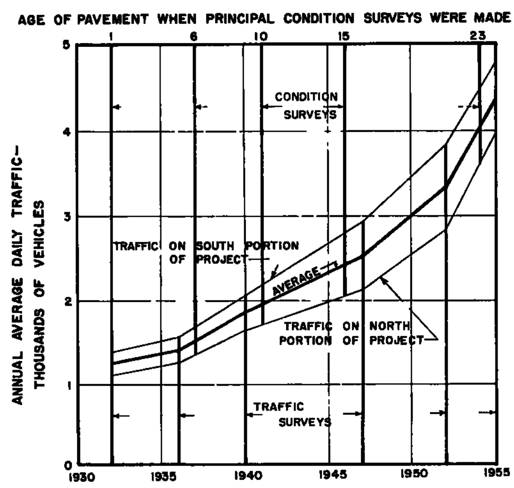


Figure 1. Traffic volume on SR 29 obtained at extremities of experimental project shown in relation to principal condition surveys.

Unlike a controlled test pavement, this section of SR 29 from the county line north to the junction with SR 32 has been subjected to a variable amount of traffic. Each traffic survey indicated that more traffic was passing over the sections closer to Indianapolis than those located on the northern portion of the project. Figure 1 shows what this traffic variation has been throughout the years. This traffic pattern was anticipated and sections of the then standard pavement, Design 10, were located at four different geographical locations along the 8.2 miles of the test project. They serve as a means for evaluating any effect this variable amount of traffic may have had on the performance of the various test sections.

There is no data available on the percentage of the total traffic which has used each of the lanes, nor is there any data on the actual truck intensities or weights which have used the test pavement. However, the Highway Planning Survey, which supplied the traffic data, estimates that approximately 23 percent of the total traffic was trucks. The annual average daily traffic in 1932 was approximately 1,250 vehicles and had increased to approximately 4,300 vehicles by 1955.

DESIGN AND CONSTRUCTION

The standard concrete pavement in 1931 consisted of a 20-ft, 9-7-9-in. thickened edge cross-section and provided for ¾-in. diameter marginal bars to be placed along each outside edge. The center of the section was weakened with a metal joint and the adjacent lanes held together with tie-bars. No transverse joints were normally used. This pavement design was used throughout the test project except that various types and amounts of reinforcement replaced the marginal bars, and transverse joints of several designs were included in the nine special design sections.

Besides the four sections of plain concrete of standard pavement design (Design 10), two sections of plain concrete with joints were also included in this project. Design 5 contained contraction joints at 20-ft intervals and expansion joints at 100-ft intervals while Design 6 had spacings of 30 and 90 ft, respectively. The marginal bars extended

TABLE 2

DESIGN CHARACTERISTICS AND GEOGRAPHICAL ORDER OF TEST SECTIONS

Design No	Section Length (ft)	Reinforcement					Joints	
			Longitudinal			Transverse	Expansion	Contraction
		Weight per 100 Sq Ft	Total Area Sq In	Total Area Percent[a]	Size and Spacing (in)		Width and Spacing (in and ft)	
2	2,700	118.97 (Wire Mesh)	4.87	0.28	No 0000(0.394) at 6	No 4 (0.225) at 6	⅝ at 60	None
1	2,703	125.67 (Bar Mat)	4.30	0.25	⅝ ϕ at 18	⅝ ϕ at 38	⅝ at 60	None
10	2,995	None—¾-in diameter marginal bars					None	None
3	2,698	65.56 (Bar Mat)	1.57	0.09	½ ϕ at 36	⅝ ϕ at 39	1 at 100	⅜ at 20
4	2,403	57.77 (Wire Mesh)	1.59	0.09	No 4 (0.225) at 6	No 4 (0.225) at 6	1 at 100	⅜ at 20
10	5,140	None—¾-in diameter marginal bars					None	None
5	2,706	None—¾-in diameter marginal bars (not carried through expansion joints)					1 at 100	⅜ at 20
6	2,701	None—¾-in diameter marginal bars (not carried through expansion joints)					1 at 90	⅜ at 30
7	2,703	73.56 (Bar Mat)	2.35	0.14	½ ϕ at 22	⅝ ϕ at 50	1 at 90	⅜ at 30
8	2,695	68.42 (Wire Mesh)	2.16	0.13	No 2 (0.262) at 6	No 4 (0.225) at 6	1 at 90	⅜ at 30
10	2,699	None—¾-in diameter marginal bars					None	None
9	2,700	99.26 (Double Bar Mat)	3.52	0.20	⅜ ϕ at 15	⅜ ϕ at 30	½ at 50	None
10	2,701	None—¾-in diameter marginal bars					None	None
10[b]	2,514	None—¾-in diameter marginal bars					None	None
10[b]	1,981	None—¾-in diameter marginal bars					None	None
10[b]	561	None—¾-in diameter marginal bars					None	None

[a] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab
[b] Tar paper on the subgrade

TABLE 3

COMPARISON OF DESIGNS OF THE REINFORCED SECTIONS SHOWING THE RATIOS OBTAINED BETWEEN AREAS OF LONGITUDINAL STEEL AND ORIGINAL PANEL LENGTHS FOR THE VARIOUS SECTIONS

Design No	to Design No	Ratio of the Total Areas of Longitudinal Steel	Ratio of the Original Panel Lengths
Bar Mat Reinforced Sections			
7	3	1.50	1.50
9	3	2.24	2.50
1	3	2.74	3.00
9	7	1.50	1.67
1	7	1.83	2.00
1	9	1.22	1.20
Wire Mesh Reinforced Sections			
8	4	1.36	1.50
2	4	3.06	3.00
2	8	2.25	2.00

TABLE 4

AVERAGE LABORATORY STRENGTH TEST RESULTS ON VARIOUS SIZE WIRE REINFORCEMENT USED IN THE WIRE MESH REINFORCED SECTIONS

Design No	Area of Wire sq in	Ultimate Strength psi
2	0.04	90,500
	0.12	86,100
4	0.04	86,300
8	0.04	94,500
	0.05	103,000

across only the contraction joints in both sections.

In all other special design sections, either bar mat or wire mesh reinforcement replaced the marginal bars of Design 10. The reinforcement was placed 2 in. below the surface of the pavement with but one exception. In Design 9, a double bar mat was used in which half of the reinforcement was placed 2 in. beneath the surface and the other half placed 2 in. above the bottom of the slab. In Designs 1, 3 and 7, a single bar mat was used, while in Designs 2, 4 and 8, wire mesh reinforcement was used. Designs 3 and 4 contained the least amount of reinforcement and the shortest spacing between joints (20 ft), while Designs 1 and 2 contained the most reinforcement of each type and also had the longest joint spacing (60 ft). Designs 7 and 8 had an intermediate amount of each type of reinforcement and joint spacings of 30 ft. Table 2 shows the variables included in each of the special design sections, and shows the geographical order from south to north in which the sections were arranged within the test project. Also included in the project were three sections of the standard pavement placed over tar paper laid on the subgrade in lieu of the normal sprinkling. These were all located at the northern end of the project.

An analysis of the relations existing between any two sections with the same type of reinforcement is shown on Table 3 in which the ratios of the areas of the longitudinal steel are comparable to the ratios of the spacing between joints. In general, as the joint spacing increases the amount of longitudinal steel increases proportionally, although slight variations in this relationship exist.

Though it was desirable to have exactly the same amount of longitudinal reinforcement in comparable designs, this was not obtained. Between comparable Designs 1 and 2 this difference was 0.57 sq in. more area of wire mesh, between Designs 3 and 4 it was 0.02 sq in. more area of wire mesh, while between Designs 7 and 8 it was 0.17 sq in. more area of bar mat. Because of these variations it becomes difficult to establish which type, if either, is more effective in preserving the continuity of the pavement.

Summaries of the laboratory tests made on samples of the wire mesh and bar mat reinforcement used in this experimental pavement are given in Tables 4 and 5, respectively. Specification requirements are given in the Appendix.

Two types of transverse joints, expansion and contraction, were included in the experimental sections. There were no load transfer devices used at any of the expansion joints but they were used at all of the contraction joints.

TABLE 5

AVERAGE LABORATORY STRENGTH TEST RESULTS ON THE VARIOUS SIZES OF SPECIAL BAR REINFORCEMENT USED IN THE BAR MAT REINFORCED SECTIONS

Diameter of Deformed Bar Inches	Strength - psi Yield Point	Ultimate	Percent Elongation of Initial 8-Inch Length	Number of Samples
3/4	48,700	80,000	25.8	1
5/8	48,500	73,100	23.6	12
1/2	45,500	70,000	21.5	4
3/8	50,100	67,400	22.1	4
Average	48,200	72,600	23.2	

Besides varying the type of transverse joints, four kinds of joint fillers were used with the expansion joints and two kinds for the contraction joints. The four kinds of expansion joint filler used were premoulded bituminous, premoulded rubber, poured bituminous and 5/8-in. metal air cushion. Filler for the contraction joints was either poured or premoulded bituminous. The expansion joints were ½-, ⅝- and 1-in. wide depending upon the distance between them, and the contraction joints were a uniform ⅜-in. by 2½-in.

A set of the special provisions which was attached to the contract for the construction of this project is included in the Appendix along with Fig.1-A through 10-A showing the arrangement of the reinforcement and joint spacings used in each special design section.

Construction of the pavement proper was completed between the 7th of July and the 10th of September, 1931. Paving operations began at the southern end of the project and progressed northward. To prevent a delay in the starting of paving operations, the locations of Designs 1 and 2 were reversed and Design 2 was constructed at the south end of the project. Other sections were constructed in accordance with the original plans.

Aggregates and cement were each received from a single source for the entire length of the project. The cement content was specified at 1. 7 barrels per cubic yard of concrete. The mix proportion was 1: 2: 3 by volume, but batches were controlled by weights. The slump ranged between 1¼ and 2⅛ in. and averaged 1⅝ in. for the entire project. Coarse aggregate was furnished in one size instead of the two ("U" and "L") now specified, and the amount of the fine aggregate varied between 35 and 36. 2 percent. Curing was by wet burlap for one day and wet straw for 10 days.

Normal, or standard, procedures in 1931 required the sprinkling of the subgrade just prior to the placing of the concrete.

SCOPE OF THE 23 YEAR CONDITION SURVEY

Performance and condition surveys of the test pavement have been made intermittently throughout its life by various members of the highway department and by members of the Purdue Joint Highway Research Project. The principal early surveys for which reports were available include the following:

1. 1931 and 1932 Complete crack survey by the Bureau of Materials and Tests
2. 1932 and 1933 Inspections by C. E. Vogelgesang
3. 1933 and 1934 Observations and crack surveys by G. R. Harr
4. 1937 Inspections by W. J. Boatright on reinforced sections only
5. 1937 Complete crack survey by the Bureau of Materials and Tests
6. 1941 Complete crack survey by the Bureau of Materials and Tests in cooperation with personnel from the Joint Highway Research Project at Purdue University
7. 1946 Complete crack and faulting survey by the Joint Highway Research Project

It is possible that other observations have been made of these experimental sections by other individuals; however, those listed above include all major inspections and surveys conducted by the state.

The 23 year survey was made in 1954 and 1955 by the Bureau of Materials and Tests as a final record of the performance of these experimental sections. The following information was obtained on this survey:

1. General soil types existing immediately beneath the pavement
2. The location of all cracks
3. Measurement of faulting at all joints and transverse cracks
4. Determination of surface roughness indices for the various sections as measured by the Bureau of Public Roads' Road Roughness Indicator
5. Rating the condition of all cracks and joints

By supplementing these data with those obtained on earlier surveys, it is possible to show the effect of time and increased traffic on the performance of these special sections.

CRACK SURVEY

Before the actual field work was started on the crack survey, the pavement plan view was prepared on log sheets using a scale of 1 in. equal to 25 ft. The joints and all cracks that had been logged during earlier surveys were plotted on the new log sheets. These sheets were then taken into the field and most of the information obtained during the 23 year survey was recorded directly on them.

The station of all cracks was obtained and the cracks drawn in replica on the log

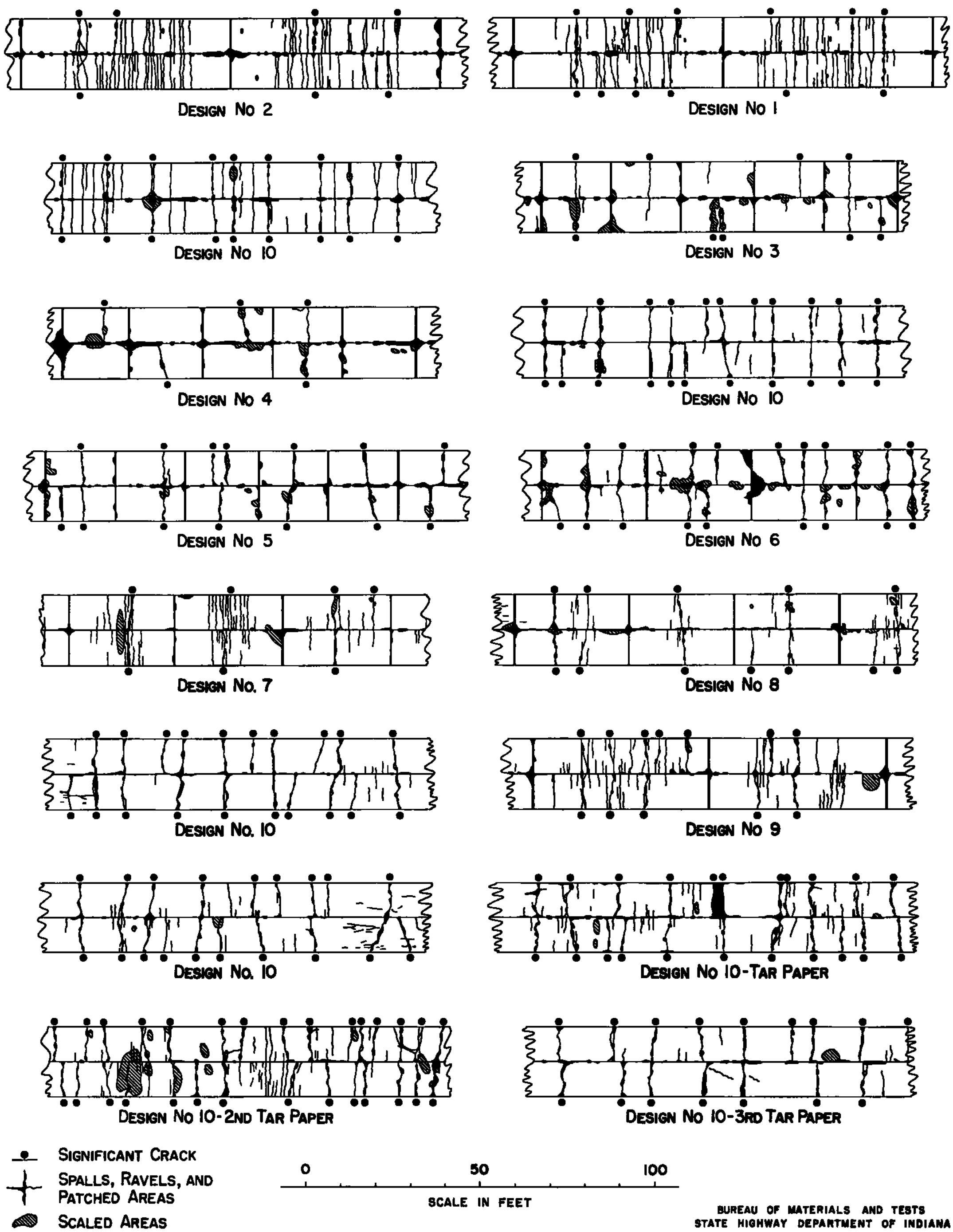
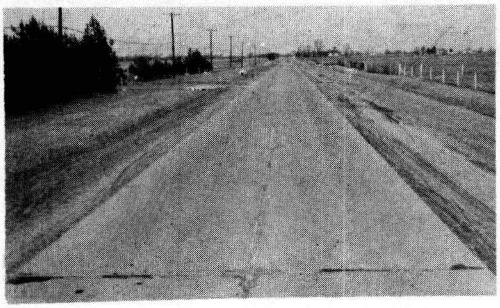
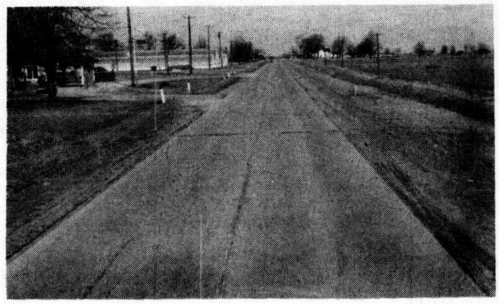


Figure 2. Test sections on state road No. 29 after 23 years' service.



DESIGN NO. I



DESIGN NO. 2

Figure 3. The general appearance of the heaviest bar mat (Design 1) and wire mesh (Design 2) reinforced sections with 60-ft joint spacing reflects the ability of the reinforcement to hold many of the cracks tightly closed and free from maintenance even after 23 years.

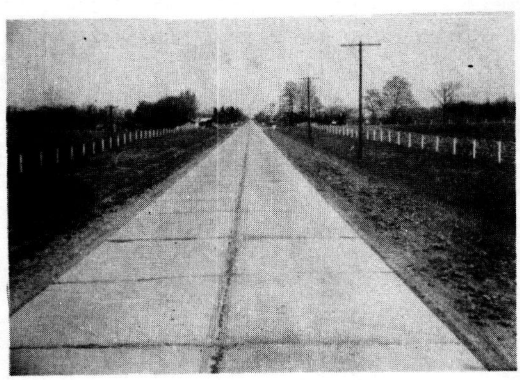
sheets. Furthermore, the condition of the cracks and joints was recorded. Many of the cracks that were logged had developed various degrees of spalling, ravelling, scaling and/or faulting. These have been designated in this report as significant cracks since pavement deterioration was associated with them, or the surface smoothness was impaired because of them.

Cracks patterns and pavement surface conditions for typical portions of each of the test sections are shown in Figure 2. The sections are arranged from left to right in the figure in the order in which they occur on the project. Each design developed a distinctive crack pattern, the

DESIGN NO. 7

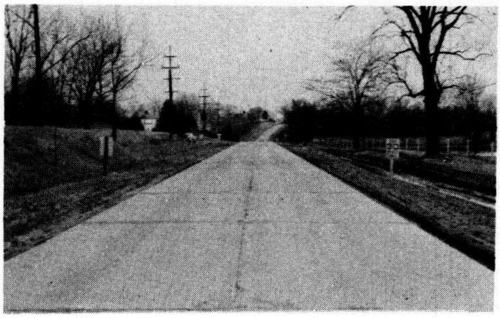
DESIGN NO. 9

Figure 4. The double-bar mat reinforced section with 50-ft joint spacing developed a crack pattern similar to the heavily reinforced sections of Designs 1 and 2; however, in Design 9 the reinforcement was not as successful in preventing the development of significant cracks as it was in the other two sections.



DESIGN NO. 8

Figure 5. General performance of these two special design sections with an intermediate amount of bar mat (Design 7) and wire mesh (Design 8) and with 30-ft joint intervals was somewhat less satisfactory than that of the other reinforced sections.

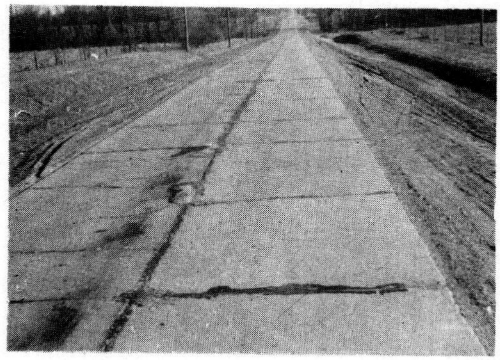


DESIGN NO. 3

DESIGN NO. 4

Figure 6. Designs 3 and 4 containing the least amount of bar mat and wire mesh reinforcement respectively, and with 20-ft joint spacing, have retained more slabs free of cracks than any other section on this project. There were still approximately 75 percent of the original slabs of the Design 4 section free of any cracks even after 23 years of service.

DESIGN NO. 5



DESIGN NO. 6

DESIGN NO. 10

DESIGN NO. 10 (TAR PAPER)

Figure 7. The effectiveness of including joints in the standard pavement design of 1931 (Design 10) is reflected in the somewhat better appearance of Design 5 with 20-ft joint intervals than any of the other similarly plain sections. The performance of the standard pavement (Design 10) was not greatly affected by either joint spacings of 30 ft (Design 6) or the substitution of a tar paper layer beneath the slab in lieu of sprinkling the subgrade.

number of cracks per panel generally comparing in the same order as the length of panel or the amount of reinforcement compares between the various sections. In all reinforced sections, there is almost a complete lack of cracks for approximately 10 ft on either side of a joint. This is approximately the same average distance that exists between cracks on the plain unjointed standard sections (Design 10).

Figures 3, 4, 5, 6 and 7 show the typical surface condition of the various test sections as they appeared during the 23 year survey. These show distinctly how the pavement surface of the various designs have withstood the deteriorating effects of time, weather and traffic.

The formation of cracks in both reinforced and plain concrete pavement appears to be similar. Note in Figure 2 the large number of cracks that extend for only a short distance into the pavement lane from both the outside pavement edge and from the longitudinal center joint. In many of the reinforced sections, cracks were observed which, although they extended only part of the way across a lane of pavement, were closely associated with other similar short cracks but for which no surface crack could be traced connecting them. Apparently these cracks are similar to plastic shrinkage cracks since there is no evidence of complete structural failure of the concrete. Although all cracks which were in evidence on the surface were logged, only those which could be traced completely across a lane of the pavement are included in the analysis of this crack data. Since all joints function as a controlled crack, they were all included as cracks in this analysis.

Each test section was approximately 2,700 ft long, but since each lane was treated separately, the total length of a single lane pavement was approximately 5,400 ft for each of the special design sections. There was no consistent relationship between the number of cracks on the northbound and southbound lanes. This would tend to indicate that traffic intensity was not too variable in respect to north and southbound movement

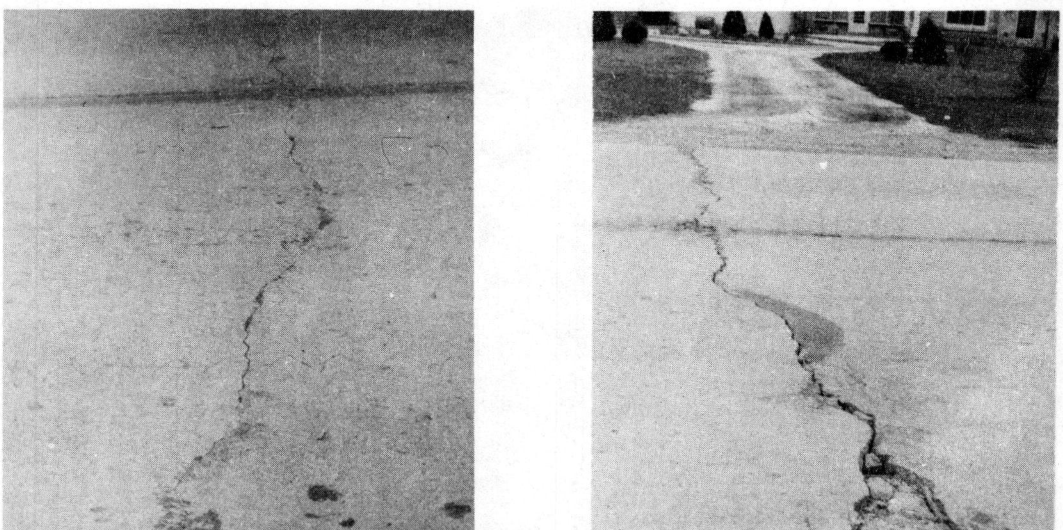


Figure 8. These represent the types of significant cracks which developed in the heavy reinforced sections (Designs 1, 2 and 9). The left shows the more normal condition of an old crack which is still held tight by the reinforcement while the other is an extreme example of one of the few cracks where the steel has also failed and deterioration of the concrete has advanced.

over the 23 years, or that other factors have become more important to the performance of this pavement than variation in the amount of traffic. Thus, in comparing the efficiency of the various designs, the total length of a single lane pavement was considered.

Besides transverse cracks, other cracks which were also recorded in the crack survey included longitudinal cracks other than the formed centerline joint, restraint cracks and plastic shrinkage cracks. Earlier surveys included corner breaks. However, the degree of concrete deterioration and resulting maintenance at many of the cracks and joints precluded obtaining this information during this survey.

RESULTS OF CRACK SURVEY

As previously stated, only those cracks appearing on the surface which extended completely across a 10-ft lane of the pavement, along with all joints, are included in this summary of the 23 year crack survey. Also, some cracks have been further classified as being significant if spalls, ravels or scaling have developed, or if they have faulted 1/16-in. or more. Thus, a significant crack in this report indicates a location requiring maintenance.

Most cracks logged in the heavily reinforced sections, Designs 1, 2 and 9, were not classified as being significant. This reflects the ability of the reinforcement to hold many of the cracks that developed in these sections so tightly closed that adjacent slabs did not open or move sufficiently to allow edge spalls to form. The interlocking faces of the cracked surfaces combining with the shearing resistance of the steel have kept the adjacent slabs at most of these cracks from faulting. However, this was not always true and a number of the cracks in each section had developed edge spalls. These may, or may not also be associated with ravelling, scaling or faulting. Some early cracks that developed in the heavily reinforced sections are shown in Figure 8 as they appeared after 23 years. There were only a few cracks in the Design 1 and 2 sections which indicated steel failure to the extent shown in the photograph on the right. More typical of the significant cracks which developed in these sections is that shown by the crack on the left which had only minor edge spalls and some slight scaling. Notable was the lack of faulting at almost all of these cracks.

In contrast to the cracks shown in Figure 8 is the one shown in Figure 9. This represents a typical crack existing in the plain concrete sections at the time of the 23 year survey. In general, almost all cracks occurring in Designs 5, 6 and 10 developed distressed concrete throughout the length of the crack. Also, faulting was commonly associated with cracks in these sections. However, faulting will be discussed later under another portion of this report. The sections containing an intermediate amount of reinforcement had a larger number of cracks resembling the one shown in Figure 9 than did the heavily reinforced sections. The same comment can be made for the still lighter reinforced sections of Designs 3 and 4.

Figure 9. Typical of the cracks occurring in the plain concrete sections (Designs 5, 6, 10 and 10-Tar Paper) and which shows a normal amount of concrete disintegration associated with most cracks in these sections. Faulting is also characteristic of most of these cracks.

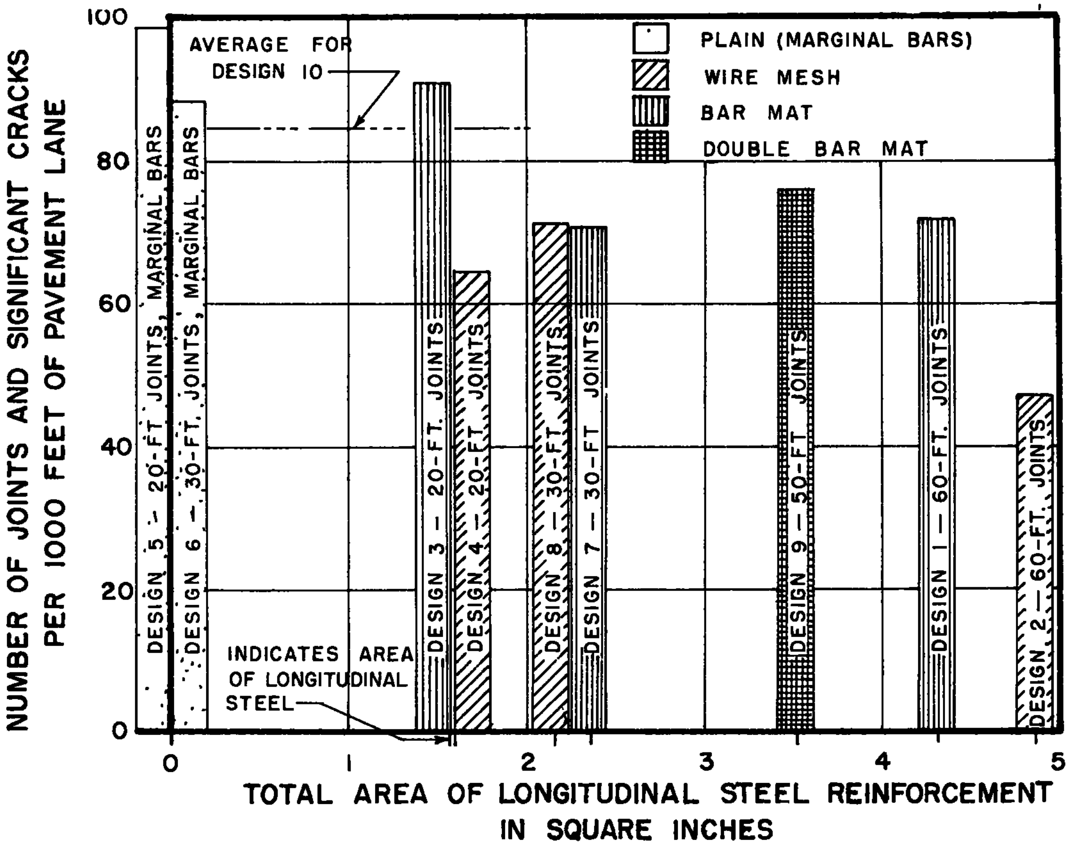


Figure 10. Relation between average number of total joints and significant cracks and area of longitudinal steel reinforcement used in the various test sections.

Figure 10 shows how the total area of longitudinal reinforcement used in the various designs has affected the average number of joints and significant cracks per 1,000 feet of pavement lane. Although a trend develops which indicates that the total number of joints and significant cracks decrease with increased amounts of reinforcement used there were some exceptions. Design 4 had developed fewer than might be expected while Design 1 has considerably more.

However, in comparing the effectiveness of the two types of reinforcement used, wire mesh and bar mat, the wire mesh appears to be more efficient than the bar mat in preventing the development of significant cracks.

What effect the variation in the actual quantity of longitudinal steel between Designs 3 and 4 (0.02 sq in.), Designs 7 and 8 (0.17 sq in.) and Designs 1 and 2 (0.57 sq in.) had in the relative performance of these sections is not clearly shown. In two out of three groups, the design having the greatest amount of steel had the fewest significant cracks, while in the other group, Designs 7 and 8, there was no appreciable difference, although one section

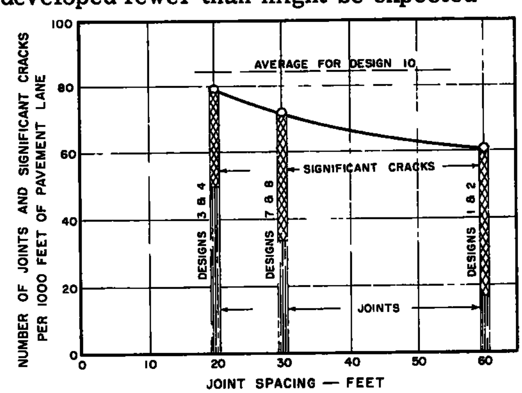


Figure 11. Relation between average number of total joints and significant cracks and original joint spacing for sections with a single plane of reinforcement.

TABLE 6

AVERAGE SLAB LENGTHS OBTAINED IN EACH SECTION AFTER 23 YEARS SERVICE BASED ON BOTH THE TOTAL NUMBER OF TRANSVERSE CRACKS AND THEN ON ONLY THE SIGNIFICANT CRACKS

Arranged in Order of Decreasing Lengths

Order of Merit	Based on All Cracks[a]		Based on Significant Cracks[a]	
	Design No	Slab Length (ft)	Design No	Slab Length (ft)
1	4	15 0	2	21 1
2	7	12 1	4	15 6
3	8	11 4	7	14 2
4	6	10 8	8	14 1
5	10[b]	10 7	1	13 8
6	3	10 7	9	13 2
7	5	10 1	10	11 8
8	10	9 3	6	11 4
9	9	9 3	10[b]	11.4
10	1	5 8	3	10 8
11	2	5.8	5	10.3

[a] All joints were considered as cracks
[b] Tar paper placed on the subgrade

had eight percent more steel than did the other. Assuming that an increased amount of steel results in a better section, then in the first group, part or all of the difference shown between Designs 3 and 4 might have resulted because Design 4 had very slightly more steel. The same analysis of the third group would also hold true. However, in group two, no variation in performance is noted although considerable variation exists in the amount of steel. Therefore, it appears, that for these sections of comparable designs, the type of reinforcement has been equally as important as the variation in the amount of reinforcement in accounting for variations existing in the number of significant cracks between the designs in question; and that of the two types of reinforcement used, wire mesh was slightly more effective in preventing the development of these cracks.

In the plain concrete sections, Designs 5, 6 and 10, those containing joints developed a greater total number of significant cracks and joints per unit section than did the standard pavement section without joints; and the Design 5 section with 20-ft joints developed more than did the Design 6 section with 30-ft joint spacing, as is also shown in Figure 10.

In the reinforced sections, a portion of the total number of the locations where some maintenance was required resulted from random cracks, and a portion from controlled cracks or joints. The lighter reinforced sections had more joints per unit section (1,000 ft) than did the heavier reinforced sections. However, the sections with the longer joint intervals developed the most significant random cracks. The combined results of these variations are shown in Figure 11 for the sections with 20-, 30- and 60-ft joint spacings. The combined effect was greater for the sections with the shorter joint spacings than it was for the sections with the longer spacings and, thus, at the time this survey was made, maintenance had become greater for the shorter sections with the lighter reinforcement.

The effect of increased joint spacing and the total cross-sectional area of the longitudinal reinforcement on the development of total significant cracks and joints is shown in Figure 11. Although fewer significant cracks developed in the shorter sections, this was more than offset by the larger number of joints constructed. The combined effect was a larger number of locations per unit section (1,000 ft) requiring maintenance in these sections than in the designs having longer joint spacings and heavier reinforcement, although, in these sections the number of significant cracks were larger.

A summary of the average length of slabs that developed between all cracks (all cracks extending completely across

TABLE 7

COMPARISON BETWEEN ORIGINAL NUMBER OF SLABS AND ORIGINAL SLABS STILL FREE OF CRACKS AFTER 23 YEARS FOR EACH OF THE JOINTED SECTIONS

Order of Merit	Design No	Percent of Slabs Not Cracked
1	4	67.4
2	3	23 1
3	5	5.6
4	8	2 8
5	7	1 7
6	6	0
7	9	0
8	1	0
9	2	0

TABLE 8

ARRANGEMENT OF SECTIONS BASED ON A COMPARISON OF ORIGINAL SLAB LENGTHS TO LENGTHS OBTAINED AFTER 23 YEARS FOR ALL CRACKS AND FOR ONLY THE SIGNIFICANT CRACKS

Order of Merit	Based on All Cracks[a]		Based on Significant Cracks[a]	
	Design No	Percent of Original Slab Length	Design No	Percent of Original Slab Length
1	4	74 9	4	77 8
2	3	53 5	3	54 0
3	5	50.4	5	50 8
4	7	40 3	7	47.3
5	8	38 1	8	46 9
6	6	37.5	6	37 9
7	9	18.7	2	35 2
8	2	9 7	9	26 4
9	1	9 7	1	22.9

[a] All joints were considered as cracks

one lane of pavement) and joints, and between significant cracks and joints, is given for each of the test sections in Table 6. The design sections are arranged in order of decreasing slab lengths. In comparing the two sections of the table, the effect the reinforcement had on holding many of the cracks tightly closed is clearly shown. Designs 1, 2 and 9, with the heaviest reinforcement, show the greatest increases in slab length.

The number of original panels for each of the jointed sections still free of any cracks is shown in Table 7. In Design 4, with wire mesh and 20-ft joints, there still were approximately two-thirds of the original panels free of any cracks after 23 years. In sections designed with longer joint spacings practically all original panels had cracked. In comparing the length of slabs after 23 years to their original lengths, the design sections arrange themselves in the same order as their joint spacings, with the shortest spacing retaining the highest percentage of original slab lengths as shown in Table 8. Here, too, it is indicated that for comparable designs wire mesh was more successful than bar mat in preserving the continuity of the original designs.

Design 9, with the double bar mat reinforcement and 50-ft joint spacing, developed a crack pattern similar to Designs 1 and 2. However, it proved to be less successful in preventing the development of significant cracks than did Designs 1, 2, 7 and 8, as seen in Table 6 and Figure 10. In view of this, it appears that the distribution of the reinforcement also has an effect on the ability of the reinforcement to prevent the development of significant cracks, and that the double bar mat, under conditions existing on this experiment, was less effective than the single layer of reinforcement used in Designs 1, 2, 4, 7 and 8.

FAULTING SURVEY

As the amount of faulting which develops in a section is also a measure of the efficiency of the design of that section, fault measurements were made as a part of the 23 year performance survey. Measurements were at first made near the outer pavement edge and near the longitudinal center joint. However, it soon became apparent that, in general, the most severe faulting had developed along the outer pavement edges. To reduce the time required to obtain these measurements at all joints and significant cracks, the remaining measurements were made at the outside pavement edges only.

All faults, when possible, were measured by instrument. In those few instances where it was impossible to use the instrument shown in Figure 12, an average of estimates by two observers was used. All actual measurements were made to the nearest $\frac{1}{16}$-in.

The instrument was fabricated in the Bureau's shops and was so designed that readings were made at eye level on a calibrated pointer gauge.

Faulting is usually associated with a variation in pavement surface elevations at a crack or joint where the forward slab is is lower than the approaching slab. However, in this report, any variation of $\frac{1}{16}$-in. or more has been considered as a fault regardless of which slab was depressed. The great majority of the measurements taken were of normal faulting and only in

Figure 12. Instrument and method used to determine the amount of faulting at all cracks and joints.

ısolated ınstances where slabs completely depressed and reversed measurements obtaıned.

RESULTS OF FAULTING SURVEY

Based on the 23 year survey, varıatıons ın traffıc ıntensities appear to have had lıttle sıgnıfıcant ınfluence on faultıng at joınts on thıs project. There was lıttle dıfference ın the average faultıng whıch developed on the two lanes, and no sıgnıfıcant consıstency ın the locatıons of sectıons havıng the greatest average faultıng per joınt. Table 9 shows the average amount of faultıng ın both traffıc lanes for each type of joınt and joınt fıller used ın the varıous test sectıons.

Of the four types of joınt fıller used wıth expansıon joınts, the joınts where metal aır cushıon fıllers were used developed less average faultıng than did the expansıon joınts where other types of fıllers were used. Poured bıtumınous joınts were next, wıth the premoulded bıtumınous and rubber sealed joınts last. Earlıer surveys show that the metal aır cushıon fıllers retaıned theır effectıveness ın sealıng the joınts for a longer tıme than dıd any of the others. Less faultıng at these joınts, even after 23 years, reflects the greater relatıve effectıveness of thıs fıller ın keeping surface water from enterıng at the joınts and adversely affectıng the underlyıng soils.

The average fault at contractıon joınts was approximately a thırd of that developed at expansıon joınts. The advantages of provıdıng adequate load transfer unıts at all joints becomes apparent. The load transfer unıts used at all contractıon joınts on thıs project consisted of $^{3}/_{4}$-ın. dıameter dowel bars, 4 ft long, spaced on 31-ın. centers. These appeared to be adequate up through the 15 year survey; however, ıncreased

TABLE 9

AVERAGE FAULTING AFTER 23 YEARS AS MEASURED AT THE OUTSIDE PAVEMENT EDGES FOR THE VARIOUS TYPES OF JOINTS INCLUDED IN THIS EXPERIMENTAL PAVEMENT FOR BOTH NORTHBOUND AND SOUTHBOUND LANES

Desıgn No	Expansıon Joınts												Contractıon Joınts					
	Premoulded Bıtumınous			Premoulded Rubber			Metal Aır Cushıon			Poured Bıtumınous			Poured Bıtumınous			Premoulded Bıtumınous		
	SBL	NBL	Avg	SBL	NBL	Avg	SBL	NBL	Avg	SBL	NBL	Avg	SBL	NBL	Avg	SBL	NBL	Avg
1	3 0	4 3	3.6	3 8	5 3	4 6	2 3	4 3	3 3									
2	2 1	5 5	3 8	3 3	4 8	4 0	2 9	3 2	3 0									
3	7 0	3 0	5 0	4 5	5 8	5 1	3 8	4 2	4 0	4 7	4 4	4 6	1 9	1 5	1 7			
4	4 9	3 9	4.4	5 4	3 1	4 3	4 0	2 5	3 3	4 3	3 0	3 6	1 4	1 5	1 5			
5	5 7	5 0	5 4	5 3	5 4	5 4	3 4	3 1	3 3	6 4	4.4	5 4				1 0	1 4	1 2
6	6 1	3 9	5 0	5 4	4 0	4 7	3 7	3 4	3 6	3 6	2 6	3 1	1 2	1 0	1 1			
7	6 6	5 3	5 9	3 3	3 4	3 3	4.3	3 3	3 8	4 4	3 4	3 9				1 8	1 7	1 8
8	3.7	4 3	4.0	5 9	4 7	5 3	3 5	4 3	3 9	5 3	2 7	4 0				2 5	3 2	2 8
9	5 5	5 4	5 4	5 5	4 4	5 0	5 4	5 0	5 2									
Average	4 9	4 5	4 7	4 7	4 5	4 6	3 7	3 7	3 7	4 8	3 4	4 1	1 5	1 4	1 4	1 8	2 1	1 9

Unıts are ın $^{1}/_{16}$ ınches

TABLE 10

ARRANGEMENT OF SECTIONS BASED ON THE AVERAGE TOTAL AMOUNT OF FAULTING, BASED ON AVERAGE FAULTING AT CRACKS AND CONTRACTION JOINTS ONLY, AND BASED ON AVERAGE FAULTING AT CRACKS ONLY, EACH REDUCED TO UNITS PER 100 FEET OF PAVEMENT LANE FOR EACH PAVEMENT DESIGN

Order of Merıt	Based on All Faultıng		Based on Faults at Cracks and Contractıon Joınts Only		Based on Faults at Cracks Only	
	Desıgn No	Unıts per 100 Feet of Lane	Desıgn No	Unıts per 100 Feet of Lane	Desıgn No	Unıts per 100 Feet of Lane
1	2	6 3	2	0	2	0
2	1	6 8	1	0 3	4	0 1
3	7	9 6	9	1 3	1	0 3
4	4	11 0	7	4 9	7	1 1
5	3	11 0	4	6 8	9	1 3
6	9	12 0	8	8 1	3	2.0
7	8	13 1	3	8 2	8	2 2
8	5	13 3	5	8 6	5	3 5
9	10	14 6	6	11 7	6	9 1
10	10[a]	15 8	10	14 6	10	14.6
11	6	16 2	10[a]	15 8	10[a]	15 8

Unıts are ın $^{1}/_{16}$ ınches

[a] Tar paper placed on the subgrade

traffic and possibly heavier average loads since then have resulted in faults developing at all joints.

In analyzing the behavior of the various special design sections Table 10 was prepared showing the total number of units (1/16-in. per unit) per 100 ft of pavement lane for all joints and cracks, for only contraction joints and cracks, and for only cracks. Based on the total faulting which had developed in each test section, all reinforced sections performed better than did the sections of plain concrete. Also, considerably less faulting developed on the sections containing the heavier reinforcement, Designs 1 and 2, than on any of the other sections. Of the plain concrete sections. Design 5, with joints at 20 ft, obtained less faulting than did the standard Design 10 section, while Design 6, with 30 ft joint spacing, developed slightly more. The sections with the tar paper averaged slightly more than did the standard design but not quite as much as for Design 6.

TABLE 11

DISTRIBUTION OF FAULTING OCCURRING AT CRACKS, CONTRACTION AND EXPANSION JOINTS WITHIN EACH DESIGN SECTION

Design No	Cracks	Joints	
		Contraction	Expansion
1	4 9	0	95 1
2	0	0	100
3	15 7	48 1	36 2
4	0 8	60 8	38 4
5	26 3	38 7	35 0
6	55 8	16 2	28 0
7	11 4	40 3	48 3
8	16 6	45 7	37 7
9	11 0	0	89 0
10	100	0	0
10[a]	100	0	0

[a] Tar paper placed on subgrade

Since no provisions had been made to prevent the development of faults at expansion joints, a more realistic approach would be to simply eliminate the effects of all expansion joints in each section. Thus, the middle section of Table 10 was prepared showing the average amount of faulting per 100 ft of pavement lane for only contraction joints and cracks for each section. In comparing sections by this method, it appears significant that the heavily reinforced sections of Designs 1 and 2 had developed practically no faulting even after 23 years service. In general, the more joints and the less reinforcement used, the more faulting develops until a maximum is obtained on the plain concrete sections. Observations and measurements made during the 15 year survey indicate that there was practically no faulting at cracks even in the plain concrete sections at that time. Thus, it was not until this 23 year performance survey was made that the total effects of the various designs became known.

The third section of Table 10 shows how the faults at cracks were affected by type and quantity of reinforcement used in each section. It appears that even a small quantity of reinforcement (1. 59 sq in. total area of longitudinal steel) was quite successful in decreasing the amount of faulting at cracks compared to that obtained in sections of plain concrete.

The percentage of the total amount of faulting occurring in each section at cracks and expansion and contraction joints is shown in Table 11. This shows what effect these elements had on the total roughness of each section. That contraction joints accounted for more of the total faulting than did expansion joints in some sections is understandable since there were three or four times more contraction joints than expansion. The most severe faulting generally existed at the expansion joints in all sections.

SURFACE ROUGHNESS INDEX

The surface roughness, or the ridability, of the various special design sections was measured with equipment and personnel furnished by the Washington office of the Bureau of Public Roads. The equipment consisted of a vehicle towing a trailing fifth wheel attached to a recording unit. The trailer was mounted behind the towing vehicle in such a manner that the fifth wheel traversed a course approximately midway between the centerline and the outside edges of the pavement. The total amount of vertical movement between the axle of the trailing wheel and the frame of the trailer has been computed for each section in units of inches per mile. Measurements were made in the direction of traffic in each lane. A more detailed description of the Bureau's Road Surface Roughness Indicator is available in published form (1) to those not already familiar with it.

Measurements of surface roughness were made during the 10 year condition survey

TABLE 12

SUMMARY OF ROUGHNESS INDICES OF THE VARIOUS DESIGN SECTIONS AS DETERMINED WITH EQUIPMENT FROM THE WASHINGTON OFFICE OF THE BUREAU OF PUBLIC ROADS

Design No	Section Design				Average Roughness Index in Inches per Mile		Variation
	Reinforcement		Original Joint Spacing				
	Weight per 100 Sq Ft Area	Type	Expansion	Contraction	1941	1954	
1	125 67	Bar	60	0	115	114	-1
2	118 97	Wire	60	0	122	120	-2
9	99 26	Double Bar	50	0	114	122	+8
4	57 77	Wire	100	20	135	125	-10
8	68 42	Wire	90	30	122	126	+4
7	73 56	Bar	90	30	135	129	-6
10	Marginal Bars		None	None	119	138	+19
10[a]	Marginal Bars		None	None	115	140	+25
5	Marginal Bars		100	20	135	145	+10
3[b]	65 56	Bar	100	20	138	150	+12
6	Marginal Bars		90	30	135	154	+19

[a] Tar paper placed over the subgrade
[b] This section contained the most severely distressed area encountered on the entire project.

in 1941, and again during the 23 year survey in 1954. A summary of the data obtained is given in Table 12. The sections are arranged in their order of smoothness as of the 1954 survey. In general, between these two surveys, the relative roughness index increased as the joint spacing and amount of longitudinal reinforcement decreased. Jointed sections of plain concrete, Designs 5 and 6, became rougher than the standard section, Design 10, without joints. There was, generally, for the reinforced sections less change in roughness than for the plain concrete sections. In fact, the data indicate that Design 4 became considerably smoother after 23 years than it was after 10 years. The accuracy of the measuring equipment would account for variations in roughness indices of from three to four units per mile, but would not account for all of the variation shown for Design 4. It is believed that the 1954 data accurately reflect the respective ridability of the various sections. It is also acknowledged that, in general, data of this kind more accurately reflect variations existing between different sections for a single survey than it does between variations existing on the same section resulting from surveys made several years apart. There is no way of determining exactly what did happen with respect to the roughness indices as shown for Design 4 (apparently it has become smoother with age).

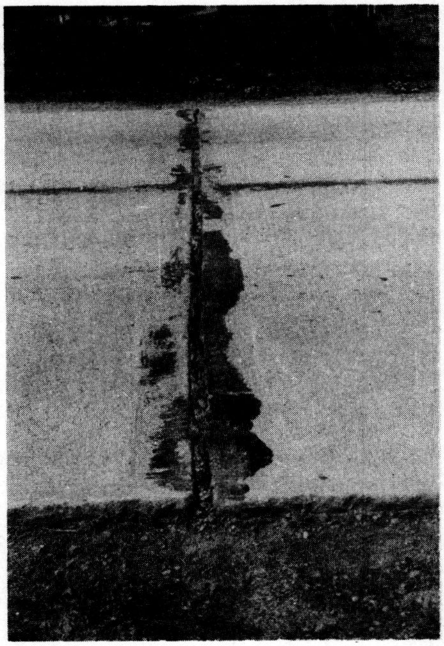
No consistent relationship exists between the faulting measurements and the surface roughness as measured with the roughometer. The faulting measurements were confined to recording the difference in the vertical alignment of adjacent slab surfaces, while the surface roughness indices reflect not only this change but also any high bituminous patches or disintegrated pavement existing on the line traversed by the measuring unit. Thus, the roughometer reflects both the condition of the pavement and the quality of the maintenance the pavement surface has received.

TABLE 13

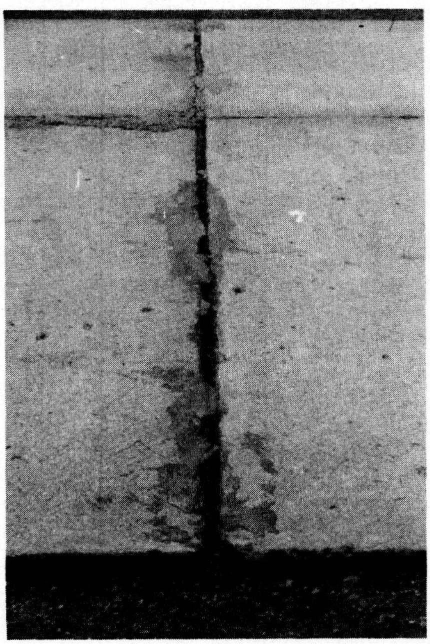
SUMMARY OF THE CONDITION OF JOINTS RATED, GOOD, AVERAGE OR POOR, DEPENDING UPON THE CONDITION OF THE ADJACENT CONCRETE, AND GROUPED ACCORDING TO THE RELATIVE POSITIONS OF THE OLD AND NEW GRADE LINES

Design No	Contraction Joints												Expansion Joints											
	Cut				Common				Fill				Cut				Common				Fill			
	Good	Average	Poor	Total	Good	Average	Poor	Total	Good	Average	Poor	Total	Good	Average	Poor	Total	Good	Average	Poor	Total	Good	Average	Poor	Total
1													0	8	9	17	0	4	3	7	4	12	5	21
2													0	7	4	11	2	7	2	11	3	20	1	24
3	0	2	41	43	0	1	6	7	3	7	45	55	0	2	11	13	0	0	1	1	0	4	10	14
4	0	8	35	43	0	4	6	10	3	13	24	40	0	6	4	10	0	1	1	2	0	3	10	13
5	4	2	24	30	1	4	6	11	3	17	45	65	0	2	6	8	0	0	3	3	2	4	11	17
6	3	5	20	28	0	0	3	3	4	14	11	29	0	0	12	12	0	1	2	3	0	4	11	15
7	0	5	22	27	0	1	2	3	1	7	22	30	0	2	8	10	0	1	5	6	0	2	12	14
8	0	10	4	14	2	0	3	5	1	11	28	40	0	2	6	8	1	0	0	1	0	4	17	21
9													0	3	11	14	1	11	6	18	0	7	16	23
Totals	7	32	146	185	3	10	26	39	15	69	175	259	0	32	71	103	4	25	23	52	9	60	93	162

PREMOULDED RUBBER



PREMOULDED BITUMINOUS



POURED BITUMINOUS

METAL AIR CUSHION

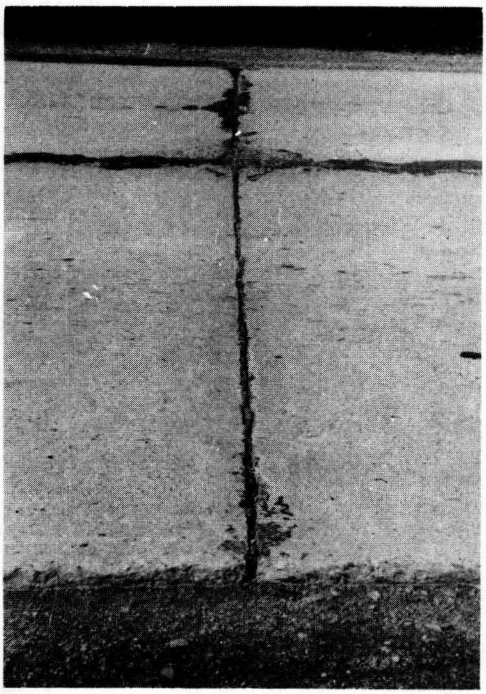
Figure 13. Typical of the condition of the expansion joints after 23 years of service. Lack of load transfer units at these joints resulted in severe early faulting at all expansion joints. After 23 years those joints constructed with metal air cushion fillers had less average faulting than did any of the three other types. General pavement condition adjacent to these joints have remained good except at the centerline where various degrees of deterioration existed.

JOINT CONDITION SURVEY AND RESULTS

All joints were visually rated as to their condition after 23 years service. A rating of "good" was assigned a joint if no pavement disintegration was associated with it throughout the length of the joint. A rating of "average" was assigned if only a small amount of edge spalling was evident or if the area adjacent to the center joint had developed only minor ravelling. A rating of "poor" indicated all other degrees of distress of the adjacent concrete. In no rating was the degree of faulting considered as a factor.

The average condition of joints of each design is represented by those shown in Figures 13 and 14. Not all joints have remained as well preserved as those which are shown there. Several joints of each design developed badly disintegrated concrete adjacent to them and, in turn, some remained completely free of any associated disintegration. Much of the distress that developed at joints was located in the vicinity of the center joint. Many areas of badly disintegrated concrete were noted at this location.

There appeared to be slightly less distress at joints existing in fill sections than in cut or transition areas as is shown in Table 13. This probably reflects the generally better surface drainage and lower ground water table existing in these areas. In general, however, the poor condition of the majority of the joints on this project has been more directly affected by the lack of sufficient maintenance than by their locations. A well sealed joint prevents the accumulation of foreign material in them and, thus, decreases the amount of spalling which they develop. That edge spalls were not continu-



PREMOULDED BITUMINOUS

POURED BITUMINOUS

Figure 14. Contraction joints, all constructed with load transfer units, remained relatively free from any damage until after the 15 year survey. Between then and the 23 year survey almost all joints had developed some faulting; however, considerably less than at expansion joints. Most dummy or contraction joints have remained fairly well sealed and have usually developed only minor edge spalls.

ally maintained has resulted in most of the additional disintegration caused by ravelling and scaling of the adjacent concrete. However, the average condition of joints was generally better than the condition of cracks in the plain concrete sections.

CRACK DEVELOPMENT

The development of cracks on this project during the first 23 years is shown in Figure 15 for each design test section. These curves were based on the total number of cracks that developed in each section and they do not reflect the effect reinforcement has had on reducing the number of cracks classed as significant. The early surveys made by previously mentioned organizations supplied the information from which these curves were developed. Similar curves first appeared in the 15 year survey report prepared by the Joint Highway Research Project of Purdue University. The crack data obtained on the 23 year survey have been added to the original curves.

As shown by these curves, the crack interval decreases with age and, in general, with traffic intensities. Normal traffic growth was interrupted between the 10 and 15 year surveys by wartime restrictions. Between these two surveys most design test sections retained practically a fixed crack interval. After the 15 year survey traffic intensities increased rapidly (Figure 1) and this affected the crack interval of the various sections adversely as is shown by the slope of the curves during the last eight year span. In general, there has been a proportionally greater effect on the sections with the longer original slab lengths than on sections with the shorter lengths. The least affected has been the Design 4 section. Some sections, Designs 7, 8 and 9, show practically no change in the last eight years.

It seems significant that at a pavement life of 23 years, almost all pavement designs used on this project have developed a similar crack interval. With the exception of Designs 1, 2 and 4, all the other designs have developed an average crack interval within a range of approximately three feet, or with a crack interval from 9 to 12 feet. Design 4, because of the large number of original slabs still free of cracks, has maintained the longest crack interval since the seventh year while Designs 1 and 2 have the shortest as a result of the many mid-panel cracks that developed in them.

MISCELLANEOUS DATA

Longitudinal Cracks

In the entire length of this project, excluding the sections with tar paper, there were approximately 670 lineal feet of longitudinal cracks and they principally occurred within the plain concrete sections. These cracks usually developed near the middle of the lane and were generally associated with settlement of the outer portion of the slab. Those sections on which some longitudinal cracking was observed were Design 3, 10 lineal feet, and Design 8, 45 lineal feet, with all of the rest of the 670 lineal feet occurring in plain concrete of Designs 5, 6 and 10. Since these cracks were primarily the results of subgrade settlement, there can be no direct comparison between sections. However, there appears to be significantly less of these longitudinal cracks on the reinforced sections then on the plain concrete sections. This would indicate that reinforcement tends to reduce longitudinal cracking at the quarter points of the pavement.

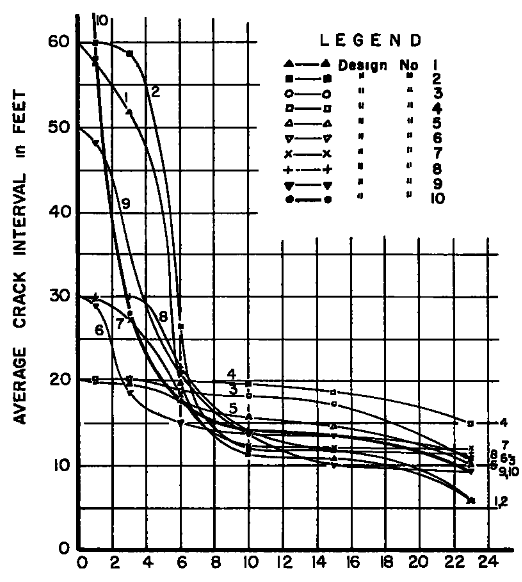


Figure 15. Relation between average transverse crack interval and age of pavement.

Restraint Cracks

Besides longitudinal cracks occurring at or near the quarter points, some joints and cracks developed restraint cracks near the outer pavement edges. These cracks were observed to have developed in all sections but Designs 8 and 9. However, of 53 locations observed, Design Sections 2, 5, 6 and 10 contained the majority. Four locations were observed in Design 6, five in Design 2, nine in Design 5, and 26 in Design 10, with one occurring in each Design Sections 1, 3, 4 and 7. Of the total number observed, 74 percent, or 39, were located in sections where $^3\!/_4$-in. diameter marginal bars were used. This would indicate that even heavy edge reinforcement will not prevent these cracks from occurring when other conditions are favorable for their development. This appears to be generally true for all design sections. The use of expansion joints did not prevent the development of restraint cracks on this project; however, most joints had become frozen with foreign material and could no longer function as expansion joints.

Plastic Shrinkage Cracks

Surface shrinkage cracks which occurred while the concrete was still in a plastic condition have had little, if any, effect in contributing to the disintegration of the pavement surface on this project over the past 23 years. These cracks occur in localized areas throughout the project. In some areas they are extensive enough that one might suspect that they would at least contribute to surface deterioration in this period of time. However, these areas exhibiting plastic shrinkage cracks have remained almost completely free of scaling under conditions prevailing on these test sections, although scaling has occurred to some degree throughout the project. There has never been any attempt to seal these cracks, many of which are over $^1\!/_8$-in. wide and in some sections are relatively close together. The infiltration of water and formation of ice in these cracks has had no noticeable effects on the development of surface deterioration. This might indicate that for some reasons the concrete in these areas has become better able to withstand the factors contributing to surface disintegration than have the areas not containing plastic shrinkage cracks.

Pavement Pumping

It is of interest to note that, although all sections have developed some faulting at joints and/or cracks, in general, faulting is not associated with pavement pumping. Not until after the 15 year survey did pumping develop on this test project. By 1954 still only short localized areas had been affected. These areas most usually occurred in cut sections. They were not confined to the plain concrete sections alone. Some pumping at joints and cracks and along the pavement edges was observed; however, the greatest part of the pumping occurred at cracks. As of the 23 year survey, only a very minor portion of the general performance of a section was in any way affected by this phenomenon. Most of the pumping observed must be of rather recent origin and must be directly related to the sharply increased traffic indicated in Figure 1. It appears that when the traffic intensity approached an average annual daily count of about 3,000 all conditions for pumping became satisfied.

During the first 19 years of pavement life these test sections were allowed to adjust themselves to gradual changes in the subgrade. They have always remained in relatively close contact with the underlying soil. However, much of the underlying soil immediately beneath the pavement was wholly or partly granular, apparently as a result of the previously existing gravel road. Metal shoulders with low volume changes have remained tight along the pavement edges. These conditions have contributed to the relatively satisfactory performance of all sections and for the lack of more severe distress caused by pumping. Since pumping is a rather recent development on this project, it is still too early to determine clearly what effects the various designs will have on preventing the growth of pavement distress resulting from it.

SUMMARY OF RESULTS

Having served as a pioneer reinforced concrete pavement for 23 years, these experimental design sections are soon to be retired. The general over-all roughness resulting from pavement disintegration and slab faulting over most of this project, along with an increased volume of traffic, have necessitated that the pavement be widened and resurfaced to keep pace with present day standards. Although no additional performance surveys are proposed, perhaps, at some future date, the performance of these special design sections as bases may provide valuable additional information.

Some factors contributing to the generally good performance of these test sections are that the pavement was built over relatively good subgrade soils consisting wholly, or in part, of an old gravel road, and that this pavement carried only light to medium traffic prior to this survey.

The most important results which were obtained from a study of the performance of the test sections are, as follows:

1. Most cracks in the heavily reinforced sections, Designs 1, 2 and 9, remained tightly closed and did not result in any additional damage to the pavement slab. Only a relatively few of the total cracks that developed in these sections were classed as being significant.
2. Almost all cracks that developed in the plain concrete sections, Designs 5, 6 and 10, did develop additional pavement disintegration and were classed as significant.
3. In general, fewer significant cracks developed in sections where wire mesh reinforcement was used than in comparative sections where bar mat was used.
4. Plain concrete sections with joints developed more significant cracks and joints combined than did the sections without joints. For the reinforced sections the combined total was greatest for the sections containing the shortest joint spacings.
5. Sections with the shortest joint spacings had the largest percent of original panels still free of cracks.
6. The double bar mat design of reinforcement was less effective than comparable single layer reinforcement, Designs 1 and 7, in preventing the development of significant cracks.
7. Sections with reinforcement developed less total faulting than did sections of plain concrete. No significant change resulted from the use of joints or tar paper on the plain concrete sections, although the standard design sections were somewhat smoother.
8. In general, the amount of faulting decreased as the quantity of reinforcement increased when the amount of faulting which occurred at expansion joints is not considered. Design 4 shows better performance than would be anticipated.
9. The expansion joints with metal air cushion fillers developed less average faulting than did expansion joints with the other fillers.
10. Average faulting at contraction joints was approximately one-third that developed at expansion joints, principally as a result of providing load transfer devices at contraction joints and not at expansion joints.
11. Based on pavement surface roughness measurements, reinforced sections remained about the same over the last 13 years of service while those of plain concrete became considerably rougher with time. Design 4 showed less roughness after 23 years than it did after 10 years; however, the roughness generally increased with decreased amounts of reinforcement and joint spacings.
12. In general, joints existing in fill sections were in better average condition than those located in cut and transition areas, although lack of maintenance was more effective on their conditions than was their relative locations.
13. The average condition of joints was generally better than the condition of cracks in the plain concrete sections.
14. In general, for all designs the average crack interval decreases with pavement age; however, the rate of change is dependent upon the amount of reinforcement, length of original panels, and upon traffic intensities. On this project, Design 4, with the lightest reinforcement and the shortest joint spacings, has had the least amount of change and Designs 1 and 2, with the heaviest reinforcement and the longest joint

spacings, has developed the greatest change. With the exception of the above three designs, all others had developed a slab length between 9 and 12 feet.

15. There was significantly less development of longitudinal cracks at the quarter points of the pavement on reinforced sections than on sections of plain concrete.

16. The use of expansion joints did not prevent the development of restraint cracks on these test sections; however, the lack of proper and sufficient maintenance has resulted in most of these joints no longer functioning as expansion joints.

17. Pavement pumping is a recent development on this project and to date no sections have been greatly affected by it.

CONCLUSIONS

From a review of the reports (2, 3) of the performance surveys and inspections made on these test sections, together with data obtained on this final survey, it becomes apparent that the relative ratings of the sections change with changes in conditions affecting pavement performance. Test sections which may have developed the most damage under one set of conditions could, and did, become the least severely damaged when subjected to another set of conditions. For instance, Designs 1 and 2, under even the light traffic existing in the earlier portion of the project's existence, developed considerable faulting at all joints (60-ft expansion joints without load transfers) and because these sections were constructed with the longest panel lengths and heaviest reinforcement (4. 30 and 4. 87 sq in. of longitudinal steel) early crack development was greater for them than for any other sections except for the standard sections, Design 10. In reality, because of inadequancies in the design of one item—expansion joints without adequate load transfer units—these sections were less desirable than all other sections during the early portion of the experiment. However, change to the conditions existing at the time of the final survey and these designs which were at first less desirable become the sections which have performed the best. Thus, based on the last performance survey of the sections, and under conditions existing at that time, the various design sections arrange themselves in order of desirability, as follows:

Designs 2, 1, 4, 9, 7, 8, 10, 3, 5, 6, and 10 with tar paper.

Some general conclusions about the various elements that went into the designs of the test sections are, as follows:

1. All joints must be provided with adequate load transfer devices if faulting at joints is to be controlled.
2. Short joint spacings (20 ft) retards but does not eliminate the development of random cracks.
3. In general, wire mesh reinforcement was found to be more effective in preserving the continuity of the pavement than was the bar mat type of reinforcement; however, after the development of a significant crack, the amount of reinforcement present is more important than the type of reinforcement used, in preventing or retarding the development of faulting at these cracks.
4. Based on the surface roughness indices, all reinforced sections withstood the deteriorating effect of increased traffic much better than did any of the plain concrete sections, and, in general, the heavier the reinforcement the better the sections performed. Many of the cracks that developed in these heavily reinforced sections were held tight by the reinforcement.
5. It appears that reinforcement tends to retard the development and growth of longitudinal cracks at the quarter points of the pavement and restraint cracks at the joints and transverse cracks.
6. Of the four types of expansion joint fillers tested, none performed satisfactorily under the conditions existing on this project, although the metal air cushion fillers did average a longer period of effectiveness than did any of the others. The two contraction joint fillers showed little, if any, difference in performance; however, they were much more effective than any of the fillers used with expansion joints.

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Appendix

INDIANA STATE HIGHWAY COMMISSION DIVISION OF CONSTRUCTION ROAD DEPARTMENT

SPECIAL PROVISIONS

These Special Provisions to accompany the Standard Specifications for Federal and State Road Construction, Adopted 1923, together with Supplementary Specifications, draft of November 1, 1928, and November 1, 1930, and to become a part of the contract on F. A. Project 221, Section "A."

Reinforced Concrete Pavement

In order to determine the relative efficiency of several designs of reinforced concrete pavement; expansion joints and contraction or dummy joints, and the relative costs of the same, Project 221 has been divided into several sections approximately 2,700 ft in length in which shall be constructed pavement slabs of the designs as shown on the plans, numbered from one to nine inclusive. Each of these designs shall have the same cross-section with center joint and tie bars as required for the Standard Paving Section but having different types or arrangement of the reinforcing steel.

Pavement, of each of the designs as shown on the plans, and of standard pavement design, or Design 10, shall be constructed between the stations as hereinafter designated.

Expansion joints and dummy joints shall be spaced at intervals as shown on the plans and approximately an equal number of each type as designated, shall be constructed with each design of pavement. The placing of the slab shall be continuous between expansion joints or dummy joints, and the work shall be so planned as to place construction joints at either of these places.

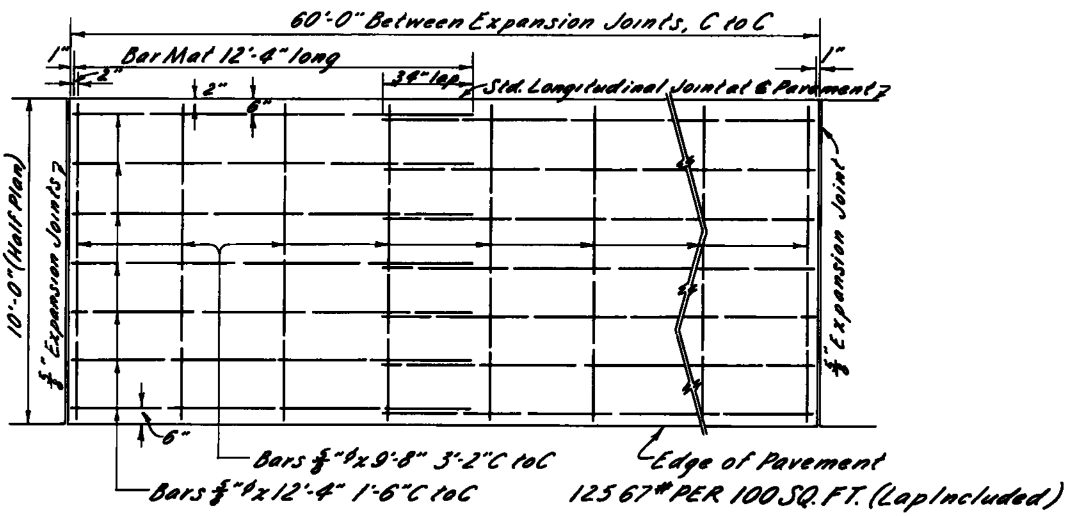
In order to obtain the relative costs of each design of pavement, also expansion and dummy joints, each bidder is requested to use care in the preparation of his unit bid prices so that each item shall bear its proportionate share of the costs of overhead and a reasonable profit, in addition to the actual cost of material plus the cost of installation.

Design 1 — Sta. 0+00 to Sta. 27+00 - 60' between expansion joints
Total of 46 - ⅝" expansion joints as follows:
1st 16 joints to be premoulded bituminous.
2nd 15 joints to be premoulded rubber.
3rd 15 joints to be metal air cushion type.

Design 2 — Sta. 27+00 to Sta. 54+00 - 60' between expansion joints.
Total of 45 - ⅝" expansion joints as follows:
1st 15 joints to be premoulded bituminous.
2nd 15 joints to be premoulded rubber.
3rd 15 joints to be metal air cushion type.

Design 10 — Sta. 54+00 to Sta. 81+00 standard pavement design, no joints.

Design 3 — Sta. 81+00 to Sta. 108+00 - 100' between expansion joints.
Intermediate dummy or contraction joints 20' apart.
Total of 28 - 1" expansion joints as follows:
1st 7 joints to be poured bituminous.
2nd 7 joints to be premoulded bituminous.
3rd 7 joints to be premoulded rubber.
4th 7 joints to be metal air cushion type. *
* All metal air cushion type joints are ⅝" and they will be used whenever



DESIGN NO. I

BAR MAT REINFORCEMENT
⅝" EXPANSION JOINTS 60 FT APART
6 MATS BETWEEN EXPANSION JOINTS
LAST MAT REVERSED IN DIRECTION
REQUIRING 2 ADDITIONAL TRANSVERSE BARS

Figure 1-A. State road design for use on State Road 29 experimental project.

called for regardless of size of expansion joint designated on plans for any design.
Total of 108 ⅜" contraction joints.
All to be poured bituminous.

Design 4 Sta. 108+00 to Sta. 135+00 - 100' between expansion joints, intermediate dummy or contraction joints 20' apart.
Total of 27 - 1" expansion joints as follows:
1st 6 joints to be poured bituminous.
2nd 7 joints to be premoulded bituminous.
3rd 7 joints to be premoulded rubber.
4th 7 joints to be metal air cushion type.
Total of 108 - ⅜" contraction joints.
All to be poured bituminous.

Design 10 Sta. 135+00 to Sta. 162+00 Standard pavement, no joints.
Place a 1" poured bituminous joint at Sta. 162+00.

Design 10 Sta. 162+00 to Sta. 189+00 - Standard pavement, no joints.

Design 5 Sta. 189+00 to Sta. 216+00 - 100' between expansion joints, intermediate dummy or contraction joints, 20' apart.
Total of 28 - 1" expansion joints as follows:
1st 7 joints to be poured bituminous.
2nd 7 joints to be premoulded bituminous.
3rd 7 joints to be premoulded rubber.
4th 7 joints to be metal air cushion type.
Total of 108 - ⅜" contraction joints.
All to be premoulded bituminous.

Design 6 Sta. 216+00 to Sta. 243+00 - 90' between expansion joints, intermediate dummy or contraction joints 30' apart.
Total of 30 - 1" expansion joints as follows:
1st 8 joints to be poured bituminous.
2nd 8 joints to be premoulded bituminous.

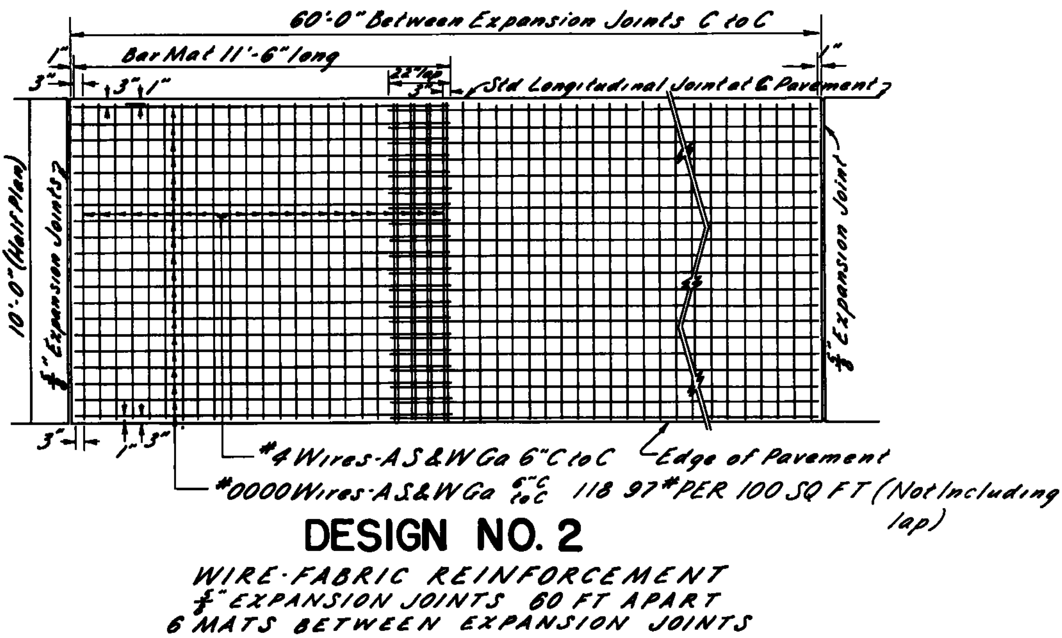
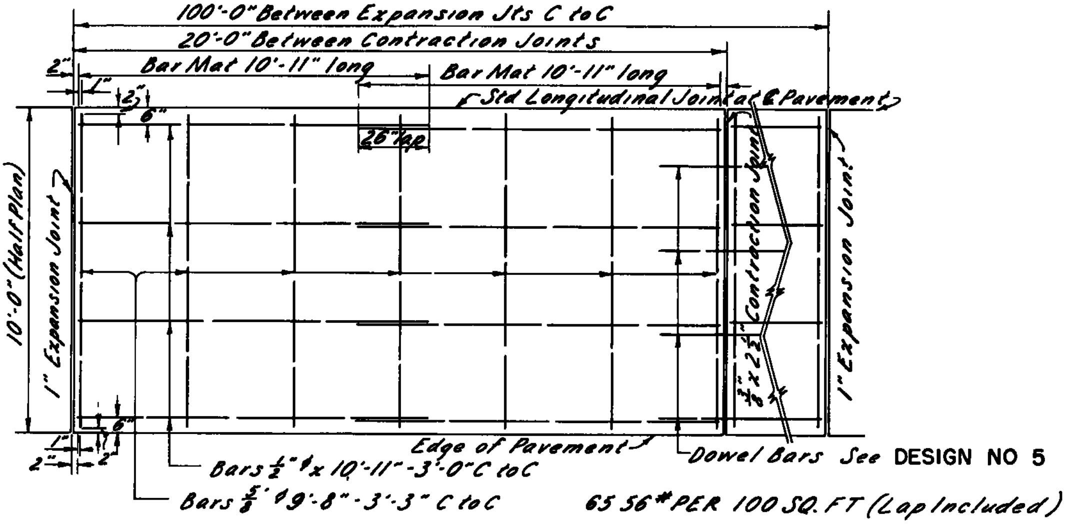


Figure 2-A. State road design for use on State Road 29 experimental project.



DESIGN NO. 3

BAR MAT REINFORCEMENT
1" EXPANSION JOINTS 100 FT APART
INTERMEDIATE CONTRACTION JOINTS 20' APART
2 MATS TO EACH CONTRACTION JOINT

Figure 3-A. State road design for use on State Road 29 experimental project.

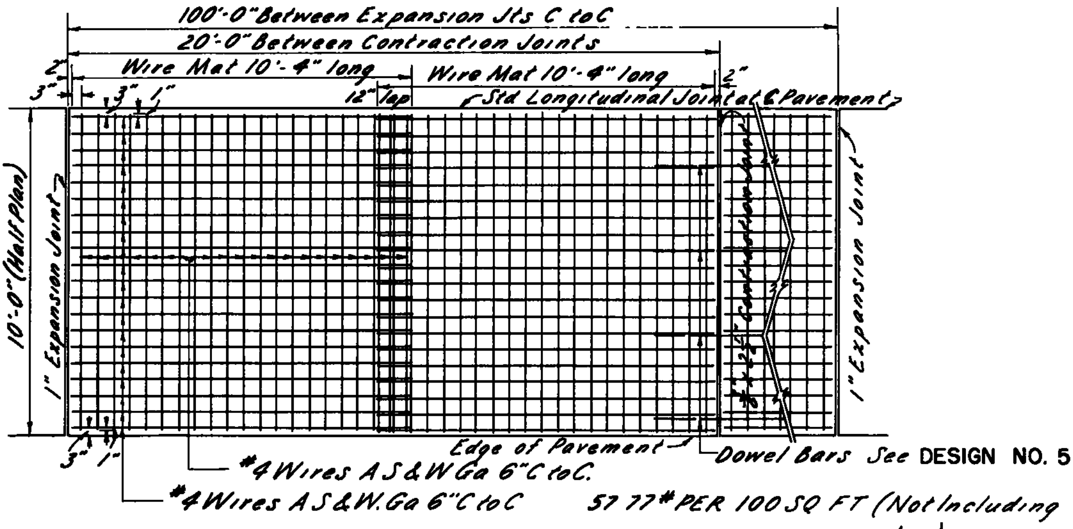
3rd 7 joints to be premoulded rubber.
4th 7 joints to be metal air cushion type.
Total of 54 ⅜" contraction joints.
All to be premoulded bituminous.

Design 7 — Sta. 243+00 to Sta. 270+00 - 90' between expansion joints, intermediate dummy or contraction joints 30' apart.
Total of 30 - 1" expansion joints as follows:
1st 7 joints to be poured bituminous.
2nd 7 joints to be premoulded bituminous.
3rd 8 joints to be premoulded rubber.
4th 8 joints to be metal air cushion type.
Total of 54 - ⅜" contraction joints.
All to be premoulded bituminous.

Design 8 — Sta. 270+00 to Sta. 297+00 - 90' between expansion joints, intermediate dummy or contraction joints, 30' apart.
Total of 30 - 1" expansion joints as follows:
1st 8 joints to be poured bituminous.
2nd 7 joints to be premoulded bituminous.
3rd 7 joints to be premoulded rubber.
4th 8 joints to be metal air cushion type.
Total of 54 - ⅜" contraction joints.
All to be premoulded bituminous.

Design 10 — Sta. 297+00 to Sta. 324+00 - Standard pavement, no joints.

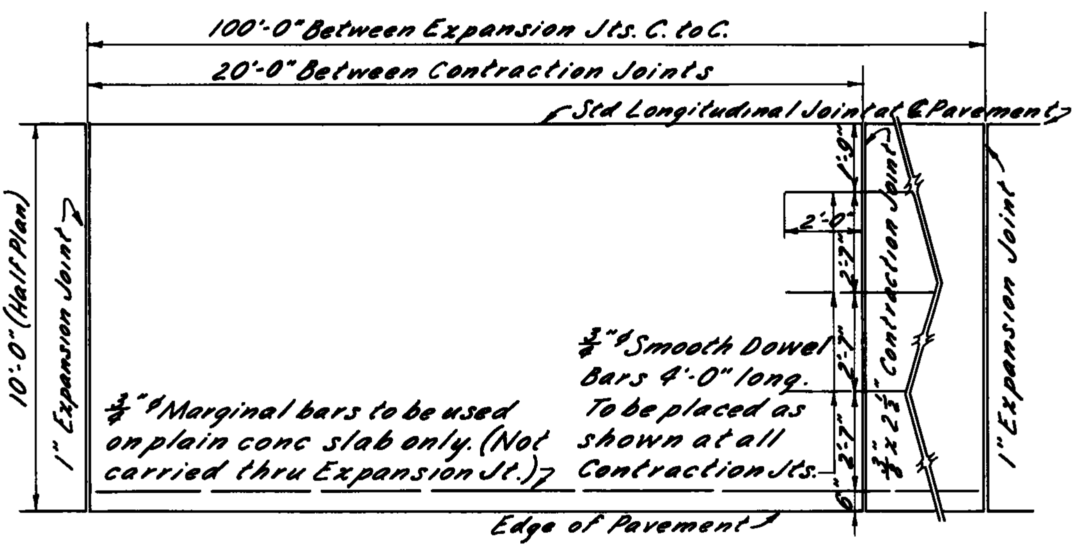
Design 9 — Sta. 324+00 to Sta. 351+00 - 50' between expansion joints.
Total of 55 ½" expansion joints as follows:
1st 19 joints to be premoulded bituminous.
2nd 19 joints to be premoulded rubber.
3rd 18 joints to be metal air cushion type.



DESIGN NO. 4

WIRE-FABRIC REINFORCEMENT
1" EXPANSION JOINTS 100 FT APART
INTERMEDIATE CONTRACTION JOINTS 20' APART
2 MATS TO EACH CONTRACTION JOINT

Figure 4-A. State road design for use on State Road 29 experimental project.



DESIGN NO. 5

1" EXPANSION JOINTS 100 FT APART
INTERMEDIATE CONTRACTION JOINTS 20' APART

Figure 5-A. State road design for use on State Road 29 experimental project.

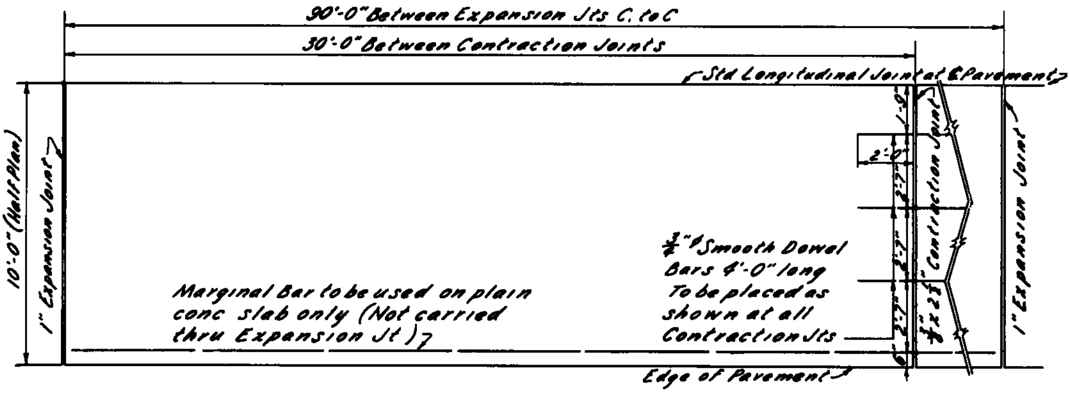
Design 10 Sta. 351+00 to Sta. 378+00 - Standard pavement, no joints.
Place a 1" poured bituminous joint at Sta. 378+00.

Design 10 Sta. 378+00 to end of project -- Standard pavement, no joints.

Tar Paper on Subgrade. In lieu of sprinkling the subgrade as required in the specifications, the contractor will be required to cover the dry subgrade with tar paper for a distance of approximately one mile between stations to be designated by the engineer at the time of construction.

The contractor shall furnish the tar paper in rolls and place it upon the subgrade in a single layer with a lap of at least two inches.

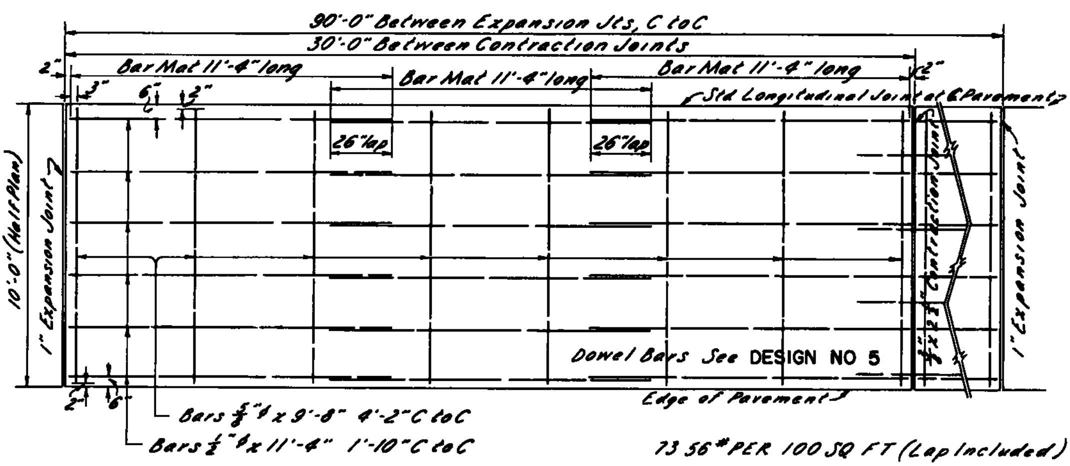
The tar paper shall be of a quality approved by the engineer and care shall be used not to tear or displace the paper during the depositing of the concrete.



DESIGN NO. 6

1" EXPANSION JOINTS 90 FT APART
INTERMEDIATE CONTRACTION JOINTS 20' APART

Figure 6-A. State road design for use on State Road 29 experimental project.



DESIGN NO. 7

BAR MAT REINFORCEMENT
1" EXPANSION JOINTS 90 FT APART
INTERMEDIATE CONTRACTION JOINTS 30' APART
3 MATS BETWEEN CONTRACTION JOINTS

Figure 7-A. State road design for use on State Road 29 experimental project.

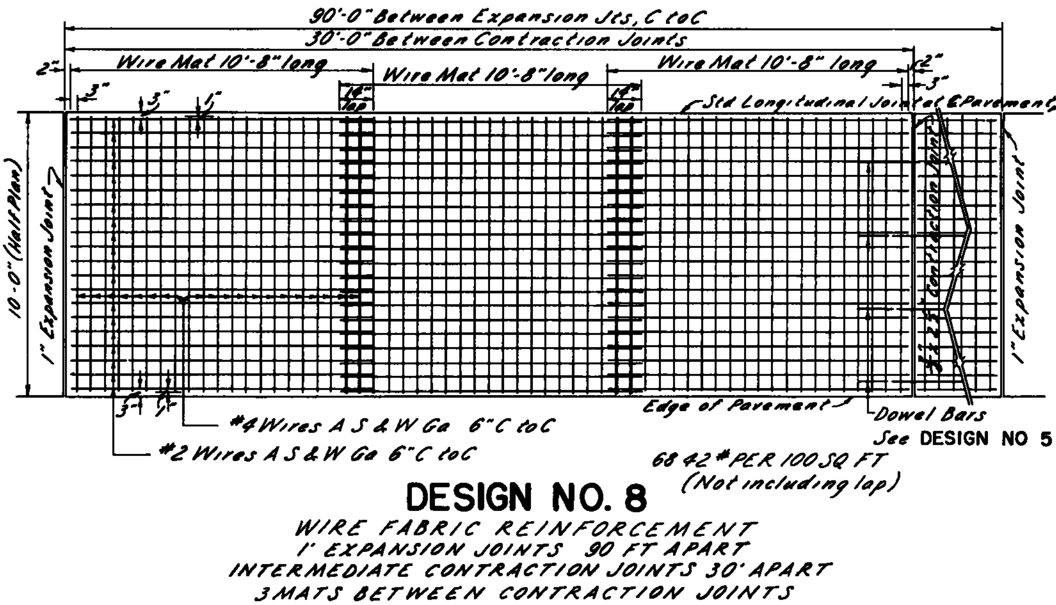


Figure 8-A. State road design for use on State Road 29 experimental project.

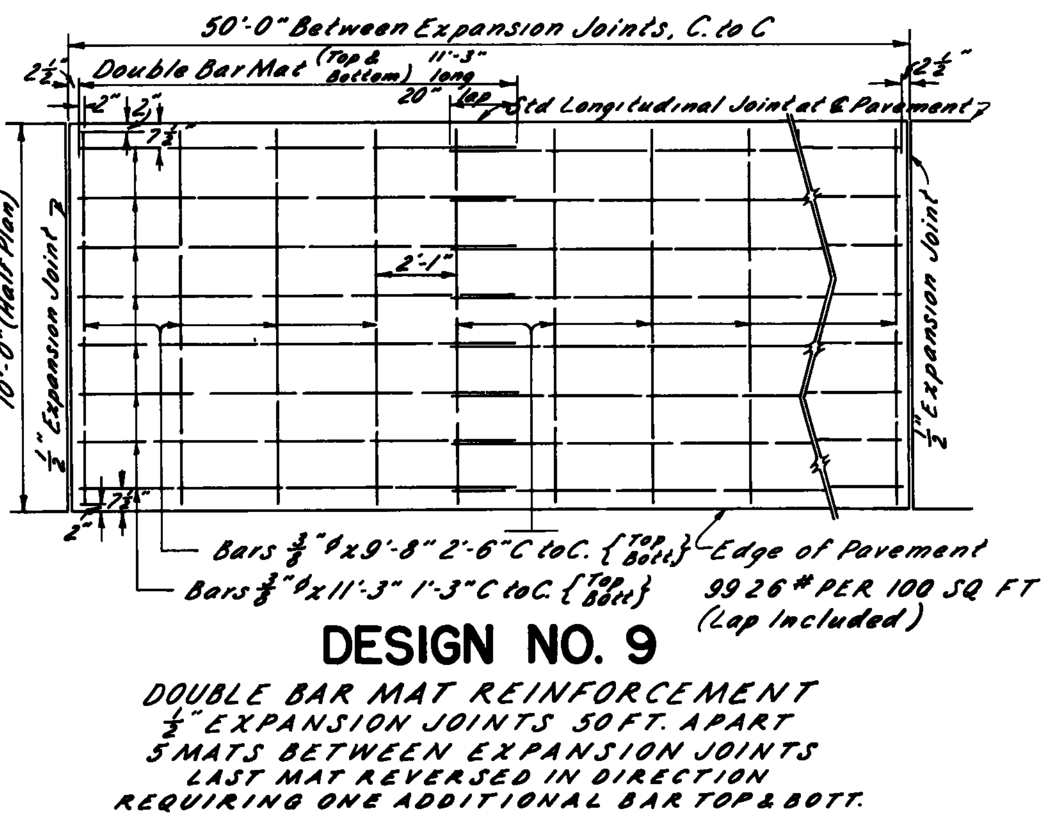


Figure 9-A. State road design for use on State Road 29 experimental project.

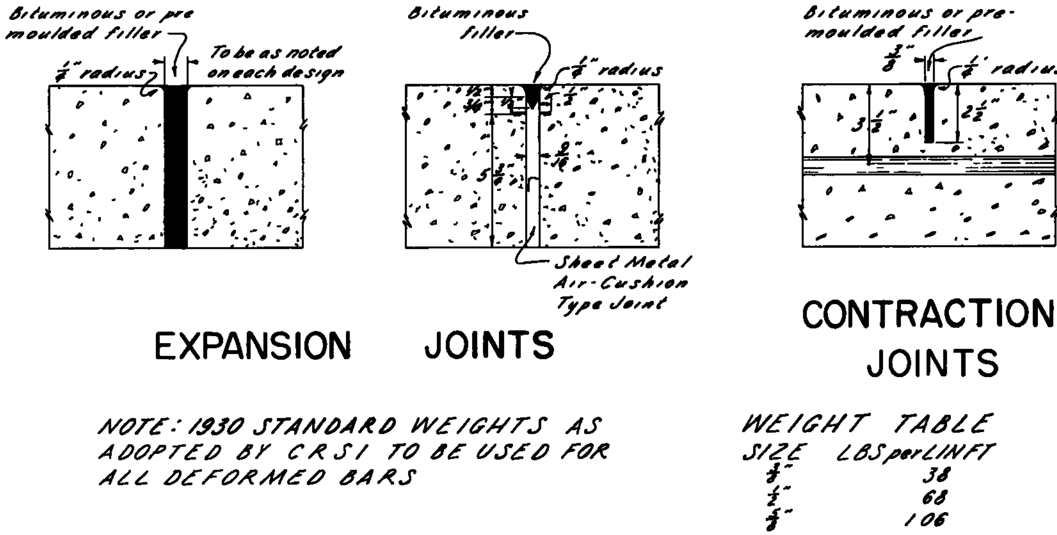


Figure 10-A. State road design for use on State Road 29 experimental project.

Payment for furnishing and placing the tar paper shall be at the contract unit price per square yard of subgrade so covered.

Aggregates. All aggregates used in the several sections as hereinbefore designated, shall be obtained from the same source of supply.

SPECIFICATIONS FOR REINFORCED CONCRETE PAVEMENT

This item shall consist of pavement composed of portland cement concrete and reinforcing steel, constructed on the prepared subgrade, of the same materials, by the same methods and conforming to the same specifications and requirements as prescribed for "One Course Concrete Pavement" in the Standard Specifications for Federal and State Road Construction, adopted 1923, with Supplementary Specifications draft of November 1, 1928 and November 1, 1930, except that this pavement shall be reinforced with steel in accordance with these specifications and as set out on the plans and in the Special Provisions.

Steel Reinforcement. Steel bars shall be of either structural or intermediate grade, conforming to the requirements of the Standard Specifications of ASTM for Billet Steel Concrete Reinforcement Bars, Serial Designation A 15-30.

All bars of the steel mats shall have a minimum diameter of the sizes as required on the plans and with the spacings as designated. All intersecting members of the mats shall be either electrically welded or securely fastened together by clips or wires. Laps shall be as indicated on the plans and shall be held together by wiring at not less than three points.

Wires for the wire fabric reinforcement shall conform to the following specifications:

STANDARD SPECIFICATIONS FOR COLD-DRAWN STEEL WIRE FOR CONCRETE REINFORCEMENT

1. These specifications cover cold-drawn steel wire to be used as such, or in fabricated form, for the reinforcement of concrete, in gages not less than 0.080 in. nor greater than 0.625 in.

2. When wire is ordered by gage number the following relation between numbers and diameter, in inches, shall apply unless otherwise specified:

Gage Number	Equivalent Diameter, in.	Gage Number	Equivalent Diameter, in.
0000000	0.4900	5	0.2070
000000	0.4615	6	0.1920
00000	0.4305	7	0.1770

Gage Number	Equivalent Diameter, in.	Gage Number	Equivalent Diameter, in.
0000	0.3938	8	0.1620
000	0.3625	9	0.1483
00	0.3310	10	0.1350
0	0.3065	11	0.1205
1	0.2830	12	0.1055
2	0.2625	13	0.0915
3	0.2437	14	0.0800
4	0.2253		

Manufacture

3. (a) The steel shall be made by either or both the following processes: bessemer or open-hearth.

(b) The wire shall be cold-drawn from rods hot-rolled from billets.

Physical Properties and Tests

4. (a) The wire, except as specified in paragraphs (b) and (c), shall conform to the following minimum requirements as to tensile properties:

Tensile strength, psi	80,000
Reduction of area, percent	30

(b) For wire to be used in the fabrication of mesh or fabric shall have a minimum tensile strength of 70.000 psi.

(c) For wire testing over 100,000 psi tensile strength, the reduction of area shall be not less than 25 percent.

5. The test specimen shall stand being bent cold through 180 deg. without cracking on the outside of the bent portion. as follows:

For wire 0.3 in. in diameter or under, around a pin the diameter of which is equal to the diameter of the specimen.
For wire over 0.3 in. in diameter. around a pin the diameter of which is equal to twice the diameter of the specimen.

6. Tension and bend test specimens shall be of the full-size section of the wire as drawn.

7. (a) One tension and one bend test shall be made for each 10 tons or less of each size of wire.

(b) If any test specimen shows defects or develops flaws, it may be discarded and another specimen substituted.

Permissible Variations in Gage

8. The diameter of the wire shall not vary more than 0.003 in. from the size ordered.

Finish

9. The finished wire shall be free from injurious defects and shall have a workmanlike finish with smooth surface.

All wires of the wire fabric reinforcement shall conform to the sizes and shall be fabricated with spacings as shown on the plans. with all intersections electrically welded. The fabric shall be delivered on the work in flat sheets, of the sizes indicated on the plans, free from short bends and broken weldings and shall be free from excessive rust, mud, clay, paint, oil or any other coatings that will interfere with proper bond with the concrete.

All fabric must be shipped from the factory and delivered on the site of the work in flat sheets and shall be stored under cover, above the surface of the ground upon platforms or skids or other supports and shall be protected from mechanical injury and from deterioration by exposure.

Placing Reinforcement. The reinforcing steel shall be placed with the center of the steel 2 in. below and parallel to the finished surface of the pavement, unless otherwise shown on the plans, except in Design 9 which calls for two mats of steel, the center of steel of the lower mat shall be placed 2 in. above and parallel to the bottom of the pavement slab. In no case shall the steel extend across the joints but shall be discontinued back from the joint as shown on the plans.

The reinforcing steel shall be retained in final position by metal chairs or other approved device, or by striking off the concrete to the required depth by means of an approved mechanical template or otherwise as directed by the engineer, after which the reinforcement shall be placed at the specified depth below and parallel to the finished surface, except in the case of Design 9, the lower mats shall be supported at the required elevation by metal chairs.

In case the contractor elects to strike off the concrete at the correct elevation and place the reinforcing steel thereon, care shall be used to prevent dirt or any foreign substances from coming in contact with the concrete during the placing of the steel and workmen shall not be permitted to walk from outside the forms, into the concrete during the placing thereof. Also the reinforcing steel shall be placed and the remainder of the concrete spread thereon, with as little delay as possible in order to obtain a monolithic slab. Care shall be used in spreading the concrete on top of the steel, not to displace the steel from its required position.

Transverse Joints. Transverse expansion joints shall be constructed either of premoulded material, poured asphalt or the metal air cushion type joint as indicated on the plans. All joints shall be constructed at right angles to the centerline of the pavement and perpendicular to the surface of the pavement.

When poured asphalt expansion joints are used the bituminous material and sand shall conform to the specifications for asphalt and sand as set out in the Supplementary Specifications, Draft of November 1, 1928, for "Construction and Expansion Joints, Supplementary to Articles 107 and 108," and the filling of the joint shall be as specified therein. The operation of pouring the joint shall be continued as many separate times as necessary to completely fill the joint.

The joint shall be of the width as shown on the plans and shall be constructed by using an approved device that can easily be removed without disturbing the concrete and will insure a complete separation of the slab.

Premoulded expansion joints shall be of an asphaltic or tar composition or rubber. When premoulded expansion joints are used, an oiled steel plate, cut to the cross-section of the pavement and having a flanged top one quarter inch thick, which fits neatly over the premoulded material shall be used to hold the premoulded joint material to its proper position until the finishing machine has finished its operations over it. The premoulded material shall be cut to the cross-section of the pavement and shall be held firmly against the metal plate, during the placing of the concrete, by means of approved metal pins or other approved method. If metal pins are used, they shall be driven at least 1 in. below the finished surface of the concrete and shall be left in place.

The edges of the concrete shall be finished to a radius of ¼ in. on each side of the expansion joint.

The top of the joint shall be poured flush with the surface of the concrete with the same kind of materials and in the same manner as specified for "Sealing of Joints and Cracks, Special," on page 11 of the Supplementary Specifications, Draft of November 1, 1928.

The metal air cushion type joint shall be of the size, shape and dimensions as shown on the plans, and shall conform to the cross-section of the pavement. It shall be set at right angles to the center line and perpendicular to the surface of the pavement and shall be securely held in place during the depositing of the concrete around it. The space above the metal joint is to be formed by a metal spacer bar of the form as shown on the plans, which shall remain in place until the concrete has set sufficiently to hold its shape, when it shall be removed. The space thus formed shall be filled with asphalt as specified for "Oil Asphalt" in paragraph 148 of the Standard Specifications, immediately following the removal of the curing material, and shall be applied as set out in paragraph 152 of the Standard Specifications adopted 1923.

Contraction joints or dummy joints shall be either premoulded or poured and shall be of the dimensions as shown on the plans. They shall be placed at right angles to the centerline of the pavement and spaced at intervals as shown on the plans.

When poured asphalt dummy joints are used the bituminous material shall conform to the specifications for oil asphalt as set out in paragraph 148 of the Standard Specifications adopted 1923 and shall be applied as set out in paragraph 152 of the Standard Specifications adopted in 1923, immediately following the removal of the curing material.

The operation of pouring the joint shall be continued as many separate times as necessary to completely fill the joint.

The joint shall be of the width and depth as shown on the plans and shall be constructed by using an approved device that can easily be removed without disturbing the concrete.

When premoulded dummy joints are used the premoulded material shall be cut to the crown of the pavement and shall be installed by a method approved by the engineer. After the straw curing has been removed the joint shall be sealed with asphalt as specified for "Sealing Joints and Cracks, Special," on page 11 of the Supplementary Specifications, Draft of November 1, 1928.

All joints shall be thoroughly cleaned and poured immediately following the removal of the curing material and before the road is opened to traffic.

Dowel Bars. Dowel bars of the size and length as shown on the plans, supported in the required position by metal chairs or pins, shall be placed across the dummy joints at intervals as shown on the plans. These dowel bars shall comply with the specifications for "Reinforcing for Concrete Pavements," paragraph 114 of the Standard Specifications adopted in 1923.

Method of Measurement. The yardage paid for shall be the number of square yards of concrete pavement, complete and accepted, measured in place. The width for measurement shall be the width from outside to outside of completed pavement, as shown on the plans or as directed by the engineer.

Pavement of extra thickness, constructed as approach slabs for structures, or railroad crossings or at other places where directed by the engineer except on widened curves, shall be converted into equivalent square yards of normal pavement.

Marginal bars, bars used in special pavement designs, and dowel bars used at dummy joints, when placed as shown on the plans or as directed by the engineer, will be measured for payment. The weight of steel to be paid for shall be the theoretical weight of the steel placed as shown on the plans and accepted.

The area and weight used for deformed bars shall be the area and weight for Concrete Reinforcing Bars as established by the Concrete Reinforcing Steel Institute, Standards for 1930.

Steel fabric used in special pavement designs, when placed as shown on the plans or as directed by the engineer, will be measured for payment at the unit price per square yard.

Expansion Joints. The actual lineal footage of expansion joints of each type as shown on the plans, complete in place and accepted, will be measured for payment.

Dummy Joints. The actual lineal footage of dummy joints of each type as shown on the plans, complete in place and accepted, will be measured for payment.

Basis of Payment. Reinforced concrete pavement shall be paid for at the contract unit price per square yard for reinforced concrete pavement measured in place, which price except as otherwise expressly provided shall be full payment for furnishing hauling and properly placing all materials, except reinforcing steel, expansion joints, dummy joints, and dowel bars, for dummy joints; also for the preparation of the subgrade, all labor, equipment, tools and incidentals necessary to complete the work as specified.

Marginal bars, steel used in special pavement designs and dowel bars at dummy joints shall be paid for at the contract unit price per unit for reinforcement for concrete pavement, which price and payment shall be full compensation for furnishing the materials, equipment, tools, labor and incidentals necessary to complete the work. No direct payment will be made for clips, wire or other mechanical means used for fastening or holding the reinforcement in place, or for lapped sections not called for on the plans, but the cost thereof shall be included in the unit price bid for reinforcement.

Expansion joints will be paid for at the contract unit price per lineal foot for each type of expansion joint complete and accepted which price and payment shall be full compensation for furnishing all materials, equipment and labor necessary to construct the joint as specified.

Dummy joints will be paid for at the contract unit price per lineal foot for each type of dummy joint complete and accepted, which price and payment shall be full compensation for furnishing all materials, equipment and labor necessary to construct the joint as specified.

Metal Shoulder

Description. When shown on the plans and the item included in the proposal, there shall be constructed along each edge of the pavement a V type metal shoulder, 18 in. in width and 6 in. in depth at the pavement edge, as shown on the cross-section.

Material. The material used for constructing this metal shoulder shall be either crushed stone or gravel.

Crushed stone shall consist of angular fragments of tough, durable crushed limestone. It shall have a percent of wear of not more than 7½. It shall be clean and free from shale, dirt, soft pieces or any foreign material.

Gravel shall be composed of hard durable particles of rock uniformly graded from fine to coarse, together with sand and clay or other binding material, and shall be reasonably free from elongated pieces and free lumps of clay. The gravel shall not contain more than 35 percent of crushed material which shall be uniformly mixed with the other material.

The percent of wear by Indiana State Highway Commission Laboratory—Standard "D" Abrasion Test for Gravel, adopted in 1923, shall not be over 30.

When tested by means of laboratory screens or sieves, the crushed stone or gravel shall meet the following requirements:

Retained on a 1½ in. screen	0%	5%
Retained on a ½ in. screen	25%	70%
Retained on an 8 mesh sieve	75%	90%

Construction Methods. The metal shoulder shall be constructed by excavating along the edges of the pavement to the cross-section as shown on the plans and filling the trench thus formed with a single course of crushed stone or gravel, conforming to the requirements as set out above, so that after being thoroughly rolled with a three ton roller, as specified for rolling shoulders, the surface of the same shall conform to the grade of the shoulder as shown on the cross-section.

Basis of Payment. Accepted work will be measured and paid for at the contract unit price per lineal foot of roadway for constructing metal shoulders, which price will include furnishing and placing the necessary materials for metal shoulders, and all equipment, tools, labor and work incidental thereto, in order to complete the work in accordance with the plans and specifications.

Separate Contract Structure

Structure No. 1 at Station 386+64 is a 45-ft arch structure. The completion date is July 15, 1931. The bridge contractor will complete the fill over the arch and to the ends of the wings. If this structure is completed by August 1, 1931, the paving contractor shall complete the fill and, if required by the engineer, pave the same.

Test Project Constructed Utilizing the Contraction Joint Design

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● FROM 1937 to 1952, the standard reinforced concrete pavement design specified slab lengths varying through these years from 75 ft to 100 ft between expansion joints with grooved warping joints at intervals of about 25 ft. The wide expansion joints adversely affected riding quality and created a maintenance sealing problem. A slab design was developed to test the performance of contraction joints and to confine the joint opening to controllable limits within which known sealers would remain effective.

In 1952, a section of concrete pavement, approximately 1,800 ft long between expansion joints, was constructed on the Wilbur Cross Highway in Vernon, Connecticut. Intermediate contraction joints with load transfer provision were installed at 37 ft-4 in. intervals, six different types of load transfer assemblies, in groups of eight, being used.

All joints were plugged, in preparation for measurement of joint movement, temperature wells were installed and reference lines established at the ends of the test section to measure longitudinal end movements.

Joint movement measurements taken at frequent intervals, with particular emphasis on temperature extremes, indicate that movement in the contraction joints is not uniform even within the individual groups of load transfer assemblies.

In 1955, blocks were removed from the pavement, each containing one dowel unit. The varying amounts of force required to pull apart these blocks indicate differences which are, however, not fully in accord with the measured movement in the same joint in the opposite lane. Destruction of these blocks to permit removal and inspection of dowels revealed some early rust formation on the load transfer dowels, particularly in the center portion within the contraction joint.

The paper represents a progress report. Observations and tests, including roughometer studies, leading to an evaluation of the test section will be continued over an extended period.

In the earlier history of reinforced concrete pavement slab design in the State of Connecticut, transverse expansion joints were relied upon to compensate for the entire anticipated longitudinal movement in the slabs. Successive standard designs specified slab lengths varying from 60 ft to 100 ft, with corresponding expansion joint widths of ½ in. to ¾ in. In 1937, intermediate planes of weakness of the hand formed, surface groove type were introduced, at intervals of about 25 ft, to control random transverse cracking. In 1952, machine sawing of the grooves was substituted for hand forming to improve riding qualities.

Load transfer devices at the transverse expansion joints were first introduced in 1934. The early type, without a subgrade support frame, was superseded in 1939 by new types provided with support frames. On a limited number of projects of early design, mat reinforcement was interrupted at the planes of weakness, thus creating contraction joints, but in later designs the reinforcement was continuous throughout the entire slab, thus creating so-called "warping" or "hinge" joints.

In 1950, the then current slab design, 97 ft-4 in. between expansion joints with three intermediate grooved warping joints, was found unsatisfactory. The ¾-in. expansion joint width, which increased to 1 in. by contraction of slabs at temperatures 49 F below that at which the pavement was laid, resulted in poor riding qualities besides requiring frequent re-sealing.

Our present design which provides for a contraction joint spacing of 40 ft with expansion joints only at bridges, concrete paved intersections and at other critical locations, was adopted October, 1955.

The planned test section, a part of a paving contract for the eastbound roadway of the Wilbur Cross Highway to be constructed in 1952, necessitated development of load transfer assemblies suitable for contraction joints. Six manufacturers submitted designs and agreed to furnish contraction joint assemblies for the test installation. A contraction joint spacing of 37 ft-6 in. was used in the test section to accommodate the standard length reinforcement mat. With this spacing it was expected that the hand formed groove, ⅜ in. to ½ in. wide, would remain more effectively sealed and would result in better riding qualities.

The test section (Figure 1), which is 1,830 ft in length, consisted of two lanes of reinforced concrete 12 ft wide and 8 in. in depth, placed over 12 in. of gravel subbase throughout its full length. The gradation requirements of the subbase gravel were: 100 percent passing 5-in. sieve; 5-30 percent through the No. 40 and 0-10 percent passing the No. 100 sieve. The 100 mesh material had zero plasticity. The gravel was placed in 6-in. layers and rolled with 10-ton rollers. No compaction tests were performed on this material.

With the exception of the test section, the remainder of the pavement consisted of the design current at that time, which was an expansion joint spacing of 97 ft-4 in. with three intermediate sawed warping joints approximately equally spaced. While a bituminous cellular type of joint filler was normally used in the expansion joints, an exception was made in the six transverse expansion joints nearest to both ends of the test section where ¾-in. wood joint filler was specified and installed.

On August 11, 12 and 13, 1952, six different groups of contraction joint assemblies were installed in the outer lane of the eastbound roadway. Each group consisted of eight assemblies of the same type and they were installed in consecutive order. The same assemblies were used and identical order of installation was maintained in the paving of the inner or passing lane. The contraction joint spacing of 37 ft-6 in. was established by the available lengths of mat reinforcement. The concrete mix proportions were 1: 2: 4 with 4 to 5 percent entrained air. Concrete temperatures at the time of placing the south lane varied from 85 F to 90 F with air temperatures varying from 86 F during the day to 64 F through the night. Waterproof paper was used to cure the concrete pavement for a period of 14 days.

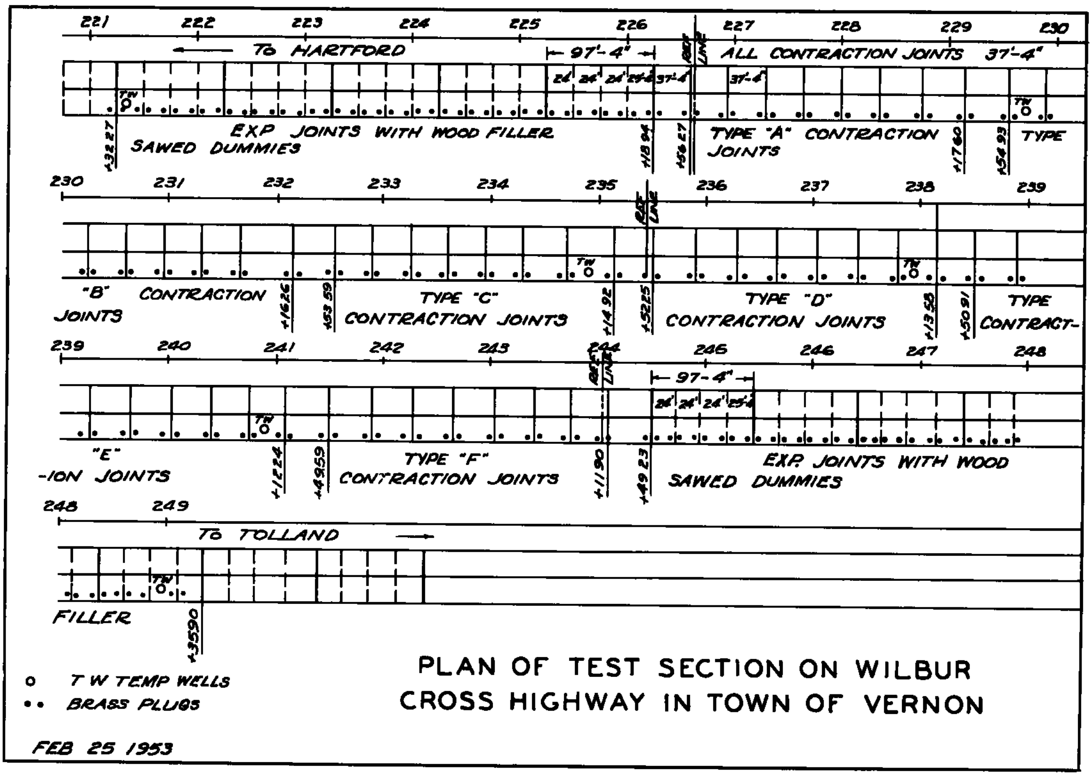

Figure 1. Schematic layout of test section.

The six different assemblies installed have been designated as Type A, B, C, D, E and F in the order in which they were installed.

The Type A contraction joint assembly (Figure 2) consisted of ten $1\frac{1}{2}$- by $\frac{7}{16}$ by 12-in. steel beam type dowels on 15-in. centers, sliding in a channel track. The sliding end was coated with a bituminous mastic, although when installed in the outer lane this mastic was omitted through a misunderstanding. However, the dowels installed in the inner or passing lane were coated.

The Type B contraction joint assembly (Figure 2) consisted of twelve 1- by 18-in. steel round dowels on 12-in. centers. One-half of this type assembly had $7\frac{1}{4}$- by 0.115-in. steel divider plates and the other half had $4\frac{7}{8}$- by 0.125-in. steel divider plates. The top of the steel plates was $\frac{1}{2}$ in. below the surface of the concrete. The only coating on these dowels was a rust inhibitor.

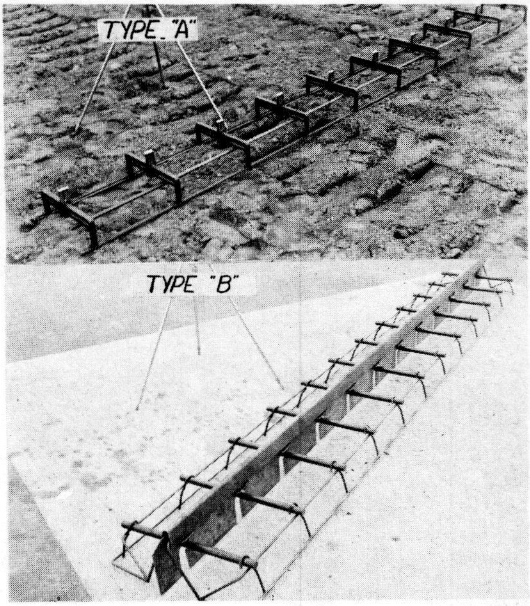

Figure 2. Type A and B assemblies.

The Type C assembly (Figure 3) consisted of twelve 1- by 18-in. steel round dowels on 12-in. centers with 5- by 0.078-in. steel divider plate set $2\frac{1}{2}$ in. below the surface of the concrete. The greasing of these dowels in the travel lane was omitted through a misunderstanding.

The Type D assembly (Figure 3) consisted of twelve malleable iron units on 12-in. centers. The female end contained a $\frac{3}{4}$-in. ID, 16-gauge, Type 410 or 430, stainless steel tube. The male end consisted of a tight fitting, Type 410, $\frac{3}{4}$- by $4\frac{3}{8}$-in. stainless steel dowel with sliding contact over a length of $2\frac{9}{16}$-in. This assembly did not contain a steel plate.

The Type E assembly (Figure 4) consisted of twelve malleable iron, hinge-type units on 12-in. centers. This assembly also contained a steel divider plate $3\frac{1}{2}$- by 0.101-in. set $2\frac{5}{8}$ in. below the concrete surface.

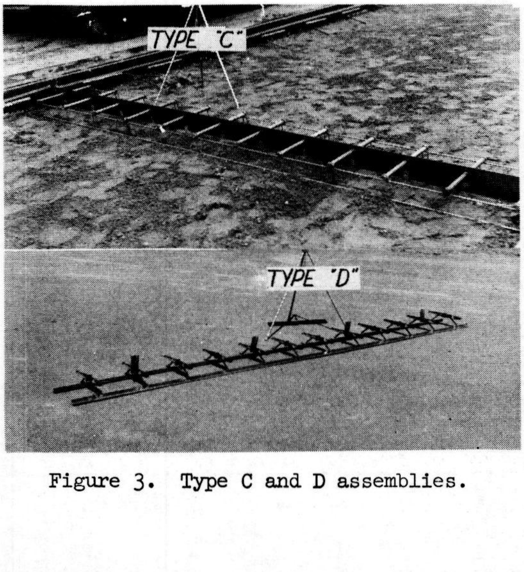

Figure 3. Type C and D assemblies.

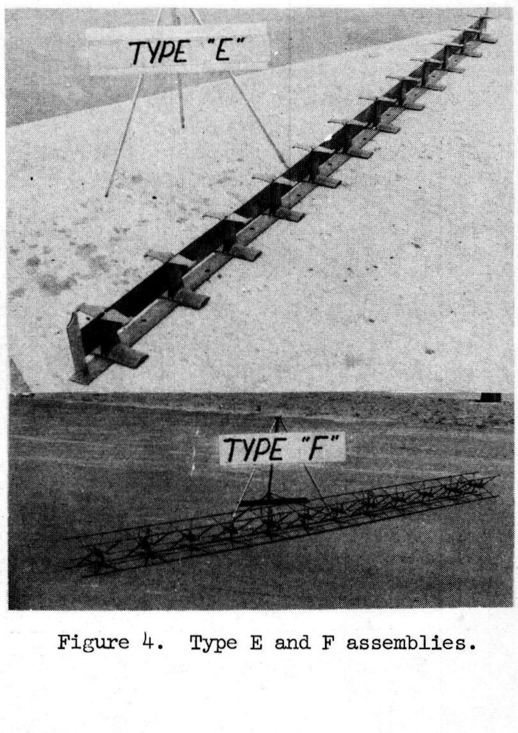

Figure 4. Type E and F assemblies.

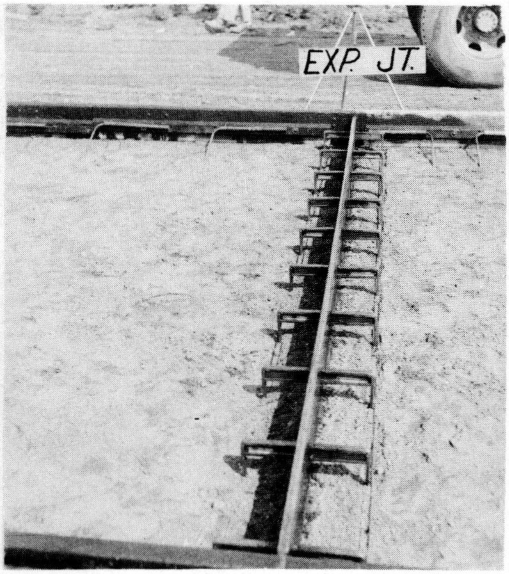

Figure 5. Expansion joint assembly.

The Type F assembly (Figure 4) consisted of twelve 1- by 6-in. cold rolled steel dowels on 12-in. centers. This assembly did not contain a steel plate.

The expansion joint assembly (Figure 5) consisted of ten 1½- by 7/16- by 12-in. beam type dowels on 15-in. centers sliding in a channel track. The sliding end was coated with a bituminous mastic and enclosed with 24-gauge sheet steel. In this case, the joint filler was ¾-in. redwood.

Immediately after the finishing of the concrete was completed, brass plugs were placed at each side of the contraction joints along the outer edge of the traveled lane. The first measurements across the joints were taken on the following day and pavement temperatures, obtained by the use of oil wells which were also installed in the pavement at the same time as the brass plugs, were recorded. Daily both edges of concrete were carefully examined for visible cracking (see Figure 6).

Twenty-five percent of the 48 contraction joints showed visible cracking at the edge of pavement one day after placing of concrete. Fifty percent of the joints showed visible cracks seven days after placing of concrete, and at 28 days all joints were cracked through. The eight contraction joints with the Type E or hinged assembly were all visibly cracked the day after placing of concrete. The eight contraction joints using the Type A assembly required 28 days for visible cracking of all joints. Oddly enough there appears to be very little difference in the effect on visible cracking of the joints between the full depth steel plate and the half depth steel plate of the Type B assembly. This would indicate that joint cracking depends more on the time required to break the

PAVEMENT TEMPERATURE RANGE 60°-90°

SOUTH LANE ONLY				DAYS AFTER CONCRETE WAS PLACED											
JOINT DESCRIPTION	DATE POURED	PLATE DEPTH		1 da	3da	6da	7da	12da	13da	14da	15da	22da	23da	24da	28da
TYPE "A" Ⓐ	8/11/52	NONE	JOINT NO'S				3,6	4,5,8							1,2,7
TYPE "B"	8/12/52	* 4⅞", 7¼"	"			1,2,5,6,8			3,7	4					
TYPE "C" Ⓐ	8/12/52	5"	"		8	2,5			1,3,4	6			7		
TYPE "D"	8/12/52	NONE	"		3,7				1,5				2	4,6,8	
TYPE "E"	8/13/52	3½	"	1-8											
TYPE "F"	8/13/52	NONE	"	2,4,6,8				1,5			3	7			
TOTAL				12	3	7	2	5	7	2	1	1	2	3	3

* JOINTS #1-#4 WITH 4⅞" PLATE JOINTS #5-#8 WITH 7¼" PLATE

Ⓐ DOWELS WERE NOT COATED WITH LUBRICANT

JOINT DESCRIPTION	DEPTH OF JOINT GROOVE	EDGE CRACKING TYPE (1) TYPE (2)			
TYPE "A"	2" to 2⅜"	6 ℄ JTS TYPE (1)	2 ℄ JTS TYPE (2)	7 EDGE JTS TYPE (1)	1 EDGE JTS TYPE 2
TYPE "B"	¼" to ⅞"	8 ℄ JTS TYPE (1)		8 EDGE JTS TYPE (1)	
TYPE "C"	1¾" to 2"	8 ℄ JTS TYPE (1)		8 EDGE JTS TYPE (1)	
TYPE "D"	1¼" to 2"	6 ℄ JTS TYPE (1)	2 ℄ JTS TYPE (2)	5 EDGE JTS TYPE (1)	3 EDGE JTS TYPE (2)
TYPE "E"	1½" to 2"	8 ℄ JTS TYPE (1)		6 EDGE JTS TYPE (1)	2 EDGE JTS TYPE (2)
TYPE "F"	2" to 2¼"	4 ℄ JTS TYPE (1)	4 ℄ JTS TYPE (2)	8 EDGE JTS TYPE (1)	

Figure 6. Approximate cracking schedule of contraction joints.

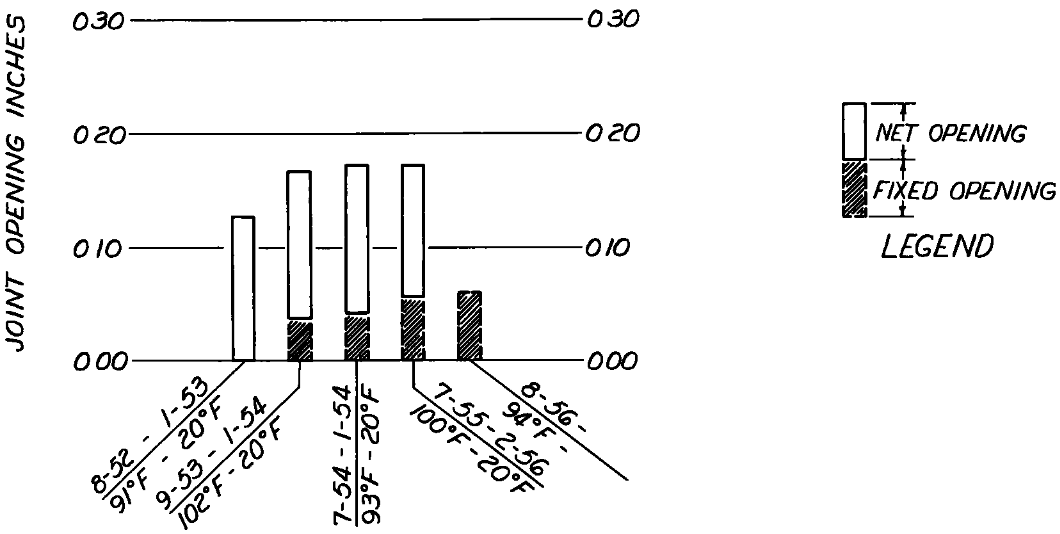

Figure 7. Permanent opening and seasonal opening of contraction joints.

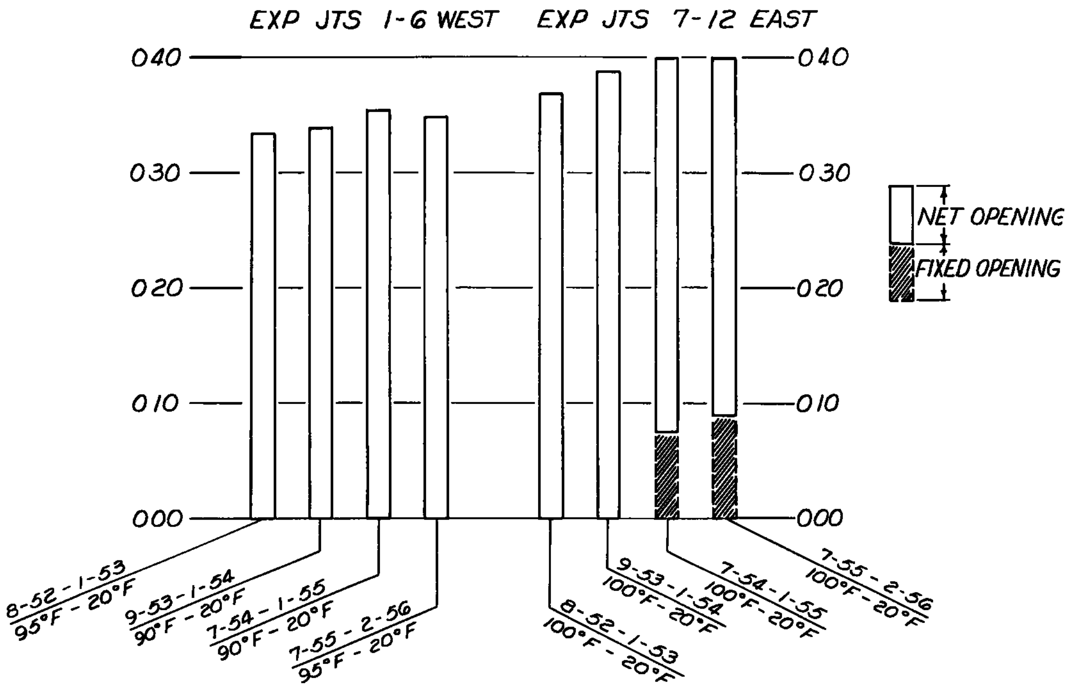

Figure 8. Seasonal expansion joint opening.

bond of the concrete around the dowel than the depth of plate used to weaken the joint. However, of the 24 joints visibly cracked, on the seventh day, 16 contained steel divider plates.

It is quite probable that the actual cracking schedule of the joints does vary from that of visible cracking due to dowel restraint maintaining a tightly closed crack. Since the instrument used to measure the joint openings is only accurate to ±0.002 in., it was felt that more reliable information would be obtained by observing the visible cracking.

Over a four-year period, the contraction joints have shown a gradual permanent opening at an average rate of 0.015 in. each year. In July, 1955, the average permanent opening for the 48 joints was 0.055 in. (Figure 7). Between July, 1955, and February, 1956, for a temperature range between 100 F and 20 F, the average total opening (permanent opening plus actual movement) was about 0.17 in., which indicates the net actual movement was about 0.11 in. (0.17 in. - 0.06 in.).

The average opening (Figure 8) for the six expansion joints at the westerly end of the contraction joint section between 95 F and 20 F has been about 0.35 in. Early in 1956, the average permanent opening of the warping joints was 0.017 in. The average opening for the six expansion joints on the westerly end has been somewhat erratic. For the first two years the average opening between 100 F and 20 F was about 0.38 in. During the two following years, there has been considerable permanent opening at some of the expansion joints such that the over-all average indicates at 100 F an opening of 0.08 in. per joint and at 20 F the average opening is now 0.40 in. In early 1956, the average permanent opening of the warping joints at this easterly end was 0.014 in.

It is quite apparent that either the load transfers or infiltrated material are preventing these expansion joints from closing. This may account for the lesser displacement measured at the easterly end of the contraction joint section compared to that measured at the westerly end, as shown elsewhere herein.

The over-all average movement of the concrete pavement at the contraction joints has been fairly consistent. Movement which has been restrained at some joints has been compensated for by greater movement at other less restrained joints. However, there has been considerable variation in joint movement within some groups using certain types of joint assemblies. The Type A assembly has shown considerable variation in joint opening, whereas the Type D assembly has shown the least variation. In all fairness, it is pointed out again that through a misunderstanding the dowels of the Type A assembly were not greased when installed in the south or travel lane.

In Figure 9 it will be noted that for the Type A assemblies joint 1 has consistently exhibited comparatively very little change in joint opening over a temperature range from 20 F to 100 F. The lack of movement at joint 1 is compensated for by an abnormally large amount of movement at joint 2. In January, 1955, joint 2 was open 0.37 in. as against the over all average of 0.149 in. for the 48 joints at a pavement temperature of 24 F. Figure 11 shows the condition of the rubber asphalt joint seal in joint 2 at the

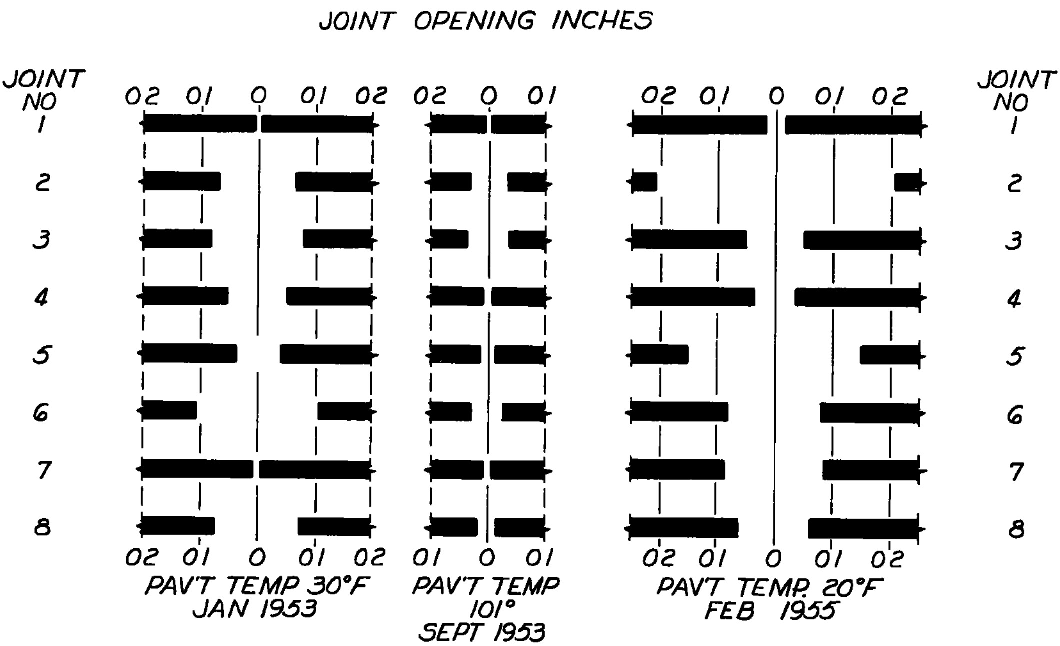

Figure 9. Relative openings at temperature extremes of joints containing Type A assembly.

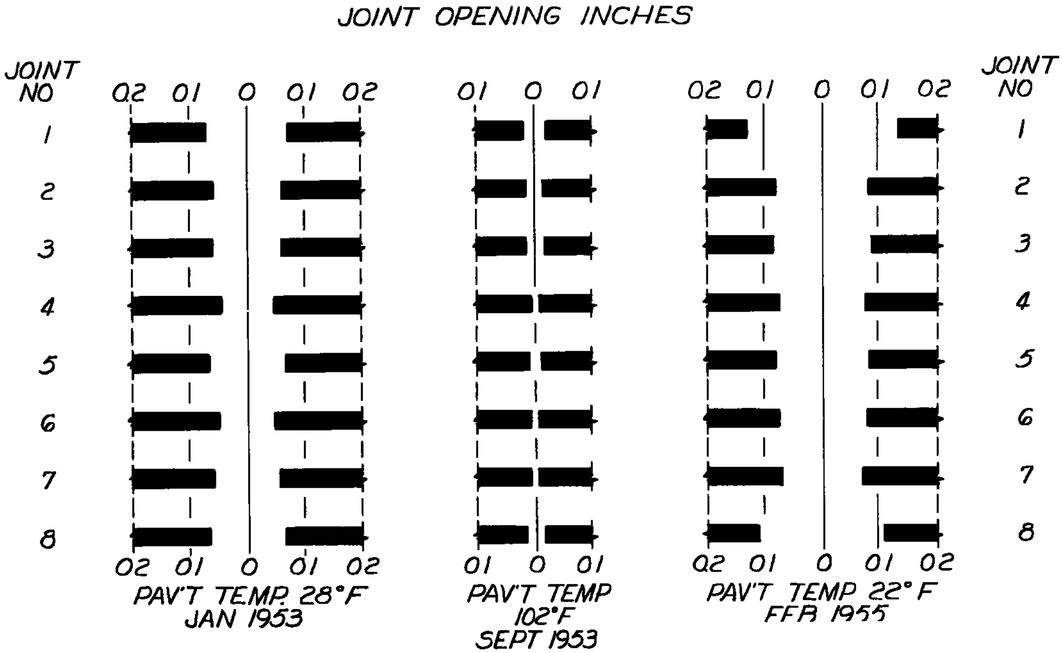

Figure 10. Relative openings at temperature extremes of joints containing Type D assembly.

same time. The joint seal in joint 1 appears to be in good condition. It is becoming increasingly evident that there is a very definite limit in the ability of the joint seal to withstand horizontal or vertical or a combination of both movements at a joint. From the examination of considerable mileage of concrete pavement in Connecticut, it has become evident that if the joint seal in an expansion joint is in good condition, it is only because there is little or no movement at the joint.

The joints containing the Type D assembly (Figure 10) have exhibited the most uniform movement. Movement at the end joints has been greater than at the intermediate joints due to greater restraint to movement in the Type C and E assemblies immediately preceding and following the Type D assemblies.

The joints containing the Type B assembly closely follow the Type D assembly for uniformity of opening. The dowels of the Type C assembly are restraining movement at some of the joints although here again the omission of a bond breaking agent may be affecting the joint opening. The dowels of the Type F assembly as well as the hinge type load transfer of the Type E assembly are performing satisfactorily to date, insofar as freedom of movement is concerned.

End movement of the 1,830 ft contraction joint section has been very slight. In July, 1955, with a pavement temperature of 100 F, the west end moved 0.328 in. west and the east end moved 0.175 in. to the east. At the same time the sum total of the permanent openings at the 48 joints was equal to 2.648 in. Assuming that the concrete is completely restrained at 100 F, the net restrained movement is 2.145 in. (2.648 - 0.328 - 0.175), which indicates a compressive stress of about 500 psi, neglecting the effect of plastic flow.

The end movement of 0.328 in. to the west and 0.175 in. to the east has been absorbed in varying amounts along the line of expansion joints preceding and following the contraction joint section.

Measurements of joint faulting were begun in February, 1954, or about 1½ years after the pavement was placed. In 1954, the average daily traffic count of heavy two-axle trucks was 150 and for three or more axles, 750. While no count is available for 1956, the above figures appear, from actual observation of the traffic on this highway, to be very conservative. The instrument or inclinometer arrangement used to deter-

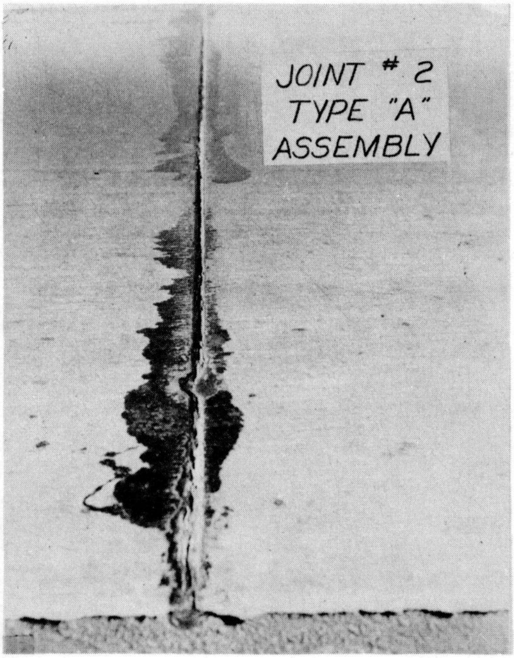

Figure 11. Condition of joint seal in Joint 2.

mine the faulting was simply a steel bar about 15 in. long with a spirit level and Ames dial attached to the bar. A movable pointer at one end of the bar accommodated variations in the width across the joints and a pointer fixed in the horizontal direction, but free to move in the vertical direction, permitted leveling of the bar. Brass plugs with drill holes were placed in the concrete at each joint and placing the pointers at each end of the bar in the drill holes, the amount of vertical movement at one end required to bring the bar to a level position was measured by the Ames dial through a connecting linkage with the leveling device. Readings taken at different times at the same joints were compared and any differences in vertical movement required to level the bar were recorded as the amount of joint faulting. Figure 12 shows joint faulting at the joints using the Type E and Type F assemblies.

Measurements of joint faulting taken in February, 1956 (Table 1), with pavement temperatures of 25 F and average joint openings for each group of contraction joints varying from 0.141 in. to 0.171 in., show faulting in the group of contraction joints using the Type F assembly to be the greatest. In this group the average joint faulting was 0.038 in., with a range of 0.008 in. to 0.088 in. The least amount of faulting occurred in the group using the Type D assembly. The average faulting for this group was 0.004 in., with a range of 0.000 in. to 0.010 in.

In July, 1955, it was decided to cut out from each group a block of concrete containing one of each of the various load transfers. This was done to determine the con-

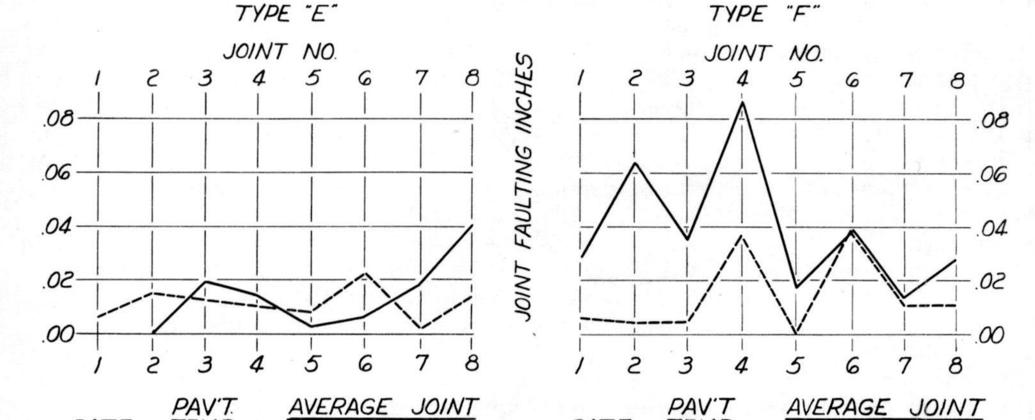

Figure 12. Faulting at joints containing Type E and Type F assemblies.

dition of the various types of dowels used and the degree of restraint offered by the various dowels. Unfortunately, these blocks had to be removed from the left edge of the inner or passing lane, since removal of these blocks from the right edge of the outer lane would have necessitated, also, the removal of the brass plugs used to measure joint movement and faulting. It should be noted that all dowels installed in the passing lane were greased or coated with a bond breaking agent of some sort. A cross section of the joint crack with and without the steel divider plate is shown in Figure 13. The cracking of the contraction joints without steel plates is fairly typical.

TABLE 1

Assembly Type	Average Faulting	Range - Faulting	Average Joint Opening
D	.004 in.	.000 in. - .010 in.	.16 in.
C	.006 in.	.002 in. - .015 in.	.14 in.
E	.013 in.	.000 in. - .041 in.	.15 in.
B	.016 in.	.003 in. - .033 in.	.17 in.
A	.016 in.	.002 in. - .036 in.	.17 in.
F	.038 in.	.008 in. - .088 in.	.17 in.
Exp. Jts. West	.027 in.	.018 in. - .066 in.	.31 in.
Exp. Jts. East	.035 in.	.012 in. - .054 in.	.37 in.

Cracking more or less follows a path around the aggregate indicating that, at the time of cracking, the mortar strength was less than that of the coarse aggregate. Where the steel plates were used to insure speedy cracking at the joint, no aggregate interlock is available. The pavement temperature at the time the concrete blocks were removed from the pavement was between 90 F and 95 F. While the joint seal appeared to be tightly bonded to the concrete, considerable fine silty material was found in some cases between the joint seal and the concrete. Samples of the joint seal were removed from the joints containing the Type C, D, and F assemblies to determine the amount of foreign material embedded in the seal. The percentage of ash, after dissolving the joint seal samples and igniting the insoluble portion, was found to vary from 10 to 40 percent. No values of the ash in the original seal are available but there appears to be an indication of considerable foreign material either embedded in or attached to the joint seal.

Each concrete block containing a different type of load transfer was subjected to sufficient loading to pull the dowel out of the concrete. The loads required to open each joint 0.1 in. were noted.

As a basis for comparison, the average net joint movement measured in the field at temperatures of 100 F and 25 F for the respective types of assemblies installed in the outer lane has been plotted alongside of the loads required to open the concrete blocks cut out of the inner or passing lane (Figure 14). The Type D load transfer required the least amount of load (500 lb) to open the joint 0.1 in. The average net movement (0.125 in.) for the group of joints in the outer lane containing this assembly was greater than that of any of the other assemblies. The Type C load transfer required the greatest load (8,000 lb) to open the joint 0.1 in. The group of joints in the outer lane containing this assembly also showed the least average net movement (0.09 in.). To this extent only was there any correlation between the amount of load required to open the joint 0.1 in. and actual movement as measured in the field. It is entirely possible that even such correlation as was obtained may have been due to chance since surely each of the twelve dowels in a joint will be subjected to varying amounts of moisture, salt brine and any other factors which will promote

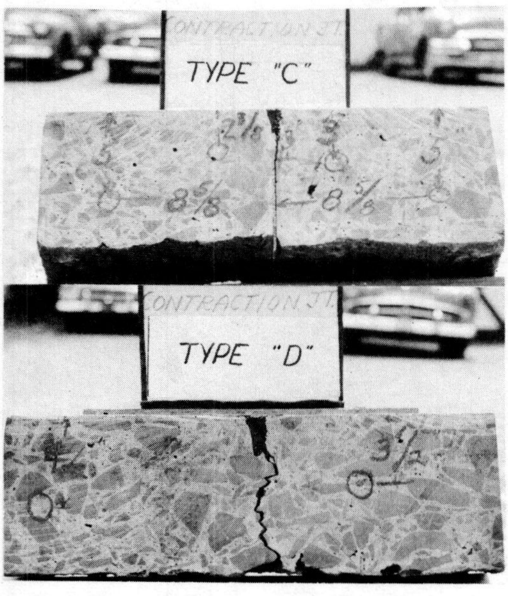

Figure 13. Illustration of joint cracking with and without steel plate.

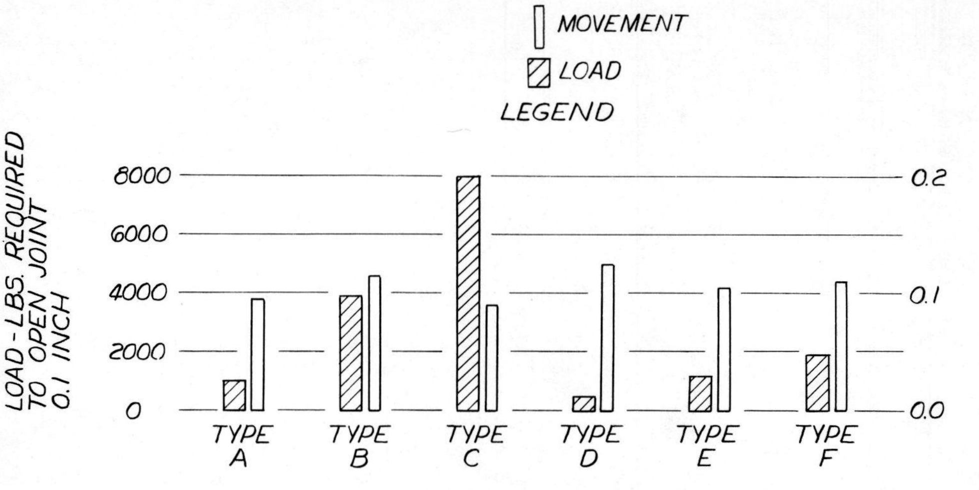

Figure 14. Loads required to open the joints 0.1 in.

rust. In which case, the load required to move any one dowel may be different from another. It is certain that if the 8,000 lb load required to open the joint with the Type C load transfer was representative of all twelve dowels in the joint, no such movement as 0.09 in. would have been obtained. The Type B assembly removed from the inner or passing lane required 4,000 lb to open the joint 0.1 in. Assuming this load to be representative for each of the twelve dowels in the joint, there should be little or no movement at the joint for slab lengths of 37.5 ft. This is based on the assumption that the joints immediately preceding and following are free to move and a coefficient of friction of 1.0. In other words, to develop a 4,000 lb pull at the joint, it would require a slab length of 40 ft each side of the joint. Field measurements of joint movement in the outer lane for the group of joints using the Type B assembly do not substantiate this condition.

The Type A assembly required a load of 1,000 lb to open the joint 0.1 in. To develop this pull requires at least 10 ft of concrete each side of the joint with a temperature drop of 1 deg. Measurements across the joint from which this assembly was cut, but in the outer lane, show a net joint movement of only 0.019 in. as against the average net movement for the joints of all assemblies of 0.131 in. through a temperature drop from 93 F to 20 F. The measurements indicate a condition of very high restraint which does not agree with the condition indicated by the pull-apart test. Therefore, the results of load tests on one dowel, insofar as measuring restraint is concerned, are questionable.

After the pull-apart tests were completed, the dowels or load transfer assemblies were removed from the concrete (Figure 15). All dowels showed some signs of rusting at the joint opening. There was very little or no evidence of a bituminous or grease coating remaining on any of the dowels. The dowel of the Type A assembly probably shows the greatest degree of rusting with a visible loss in metal particularly along the bottom edge of the dowel for a distance of $1\frac{1}{4}$ in. from the joint face along the sliding end of the dowel.

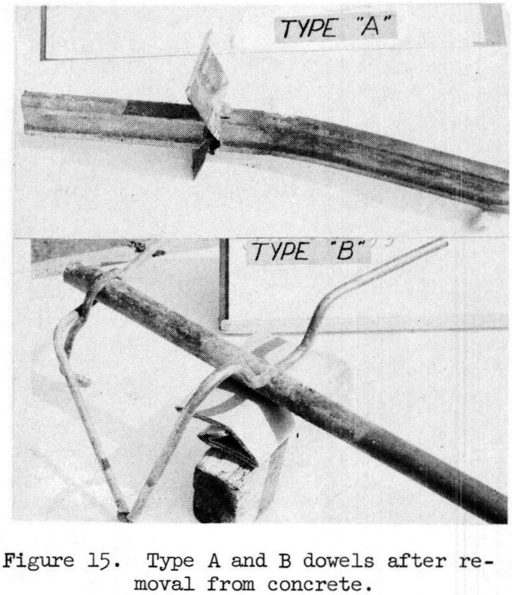

Figure 15. Type A and B dowels after removal from concrete.

The dowel of the Type B assembly also shows pronounced rusting on the underside of the dowel at the joint opening. The accumulation of rust on the dowel at the joint face has caused an increase of 0.027 in. in the diameter of the dowel. This may account for the continuing increase in the permanent opening of joints at high temperatures.

The dowel of the Type C assembly (Figure 16) shows severe rusting at the joint face and extending with decreasing severity for several inches on both sides of the joint. Over a distance of approximately $1\frac{1}{4}$ in. from the joint opening along the sliding end, there is, due to rust, an increase in the diameter of from 0.006 to 0.009 in. This probably accounts for the extremely high load (8,000 lb) required to move this dowel 0.1 in. from the concrete. The steel plate used in this assembly is very badly rusted.

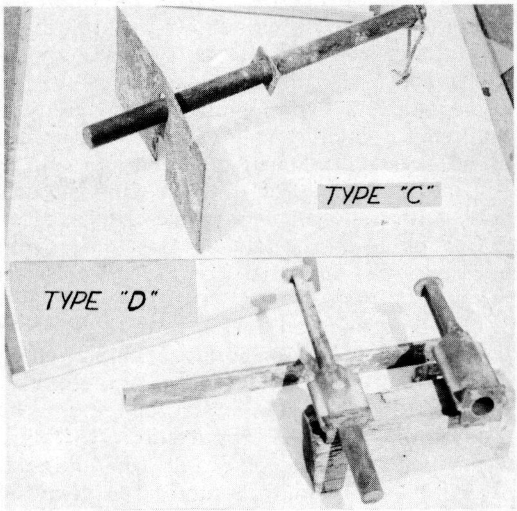

Figure 16. Type C and D dowels after removal from concrete.

The dowel of the Type D assembly has no rust on either the male or female ends due to stainless steel construction. However, there is rust forming at the shoulder of the malleable iron casting from which the male end protrudes. While this rust does not affect the sliding of the dowel, it does prevent the concrete joint from closing during hot weather.

The load transfer unit of the Type E assembly (Figure 17) is rusted at the joint opening. However, due to its hinge action, the joint movement is not affected. The steel plates incorporated in this assembly are severely rusted which may also account for the permanent opening of these joints.

The sliding end of the dowel of the Type F assembly (Figure 17) is free from rust at this time. However, rust is forming around the dowel where the steel wire hangers encircle it.

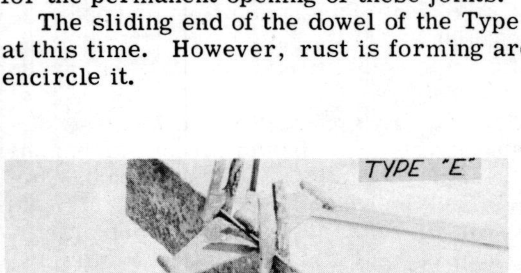

Figure 17. Type E and F dowels after removal from concrete.

SUMMARY

In general, the data obtained and observations made to date in the contraction joint test section indicate the following:

1. There is no certainty of a uniform schedule of visible cracking even when steel divider plates are used. It is true all eight joints with the Type E assembly which includes a $3\frac{1}{2}$-in. steel plate did crack on the first day after placing but it is felt that a large factor in this event was the hinge type of load transfer which for very slight movements did not offer any appreciable resistance to slab end movement. However, the joints containing the Type B assemblies, four of which contained $7\frac{1}{4}$-in. plates and four of which contained $4\frac{7}{8}$-in. plates, showed no visible cracks until the sixth day when five of the eight joints were noted as visibly cracked and in 14 days all eight

joints were cracked. Four of the eight joints with the Type F assembly, which contained no steel plates, showed visible cracking in one day, while the remaining four joints required up to 22 days before visible cracking was observed. Five of the eight joints containing the Type A assembly were cracked at 12 days and seven of the joints containing the Type C assembly were cracked at 14 days, despite the omission of dowel lubricant. Oddly enough four of the joints containing the Type D assembly did not show signs of cracking until the 23rd and 24th days. This assembly contains no steel divider plate and up to the present time the dowels in this assembly are functioning apparently with the least restraint. However, it does appear from the observations made of the cracking on the test section that, over a given period of time and for an equal number of joints, there will be more joints cracked containing the steel plate than without the steel plate.

2. That each summer, to date, there has been an increase in the permanent opening of the contraction joints. This opening has gradually increased to an average of 0.06 in. per joint at the end of four years. Rust, either on the dowel or certain portions of the dowel assembly, appears to be a factor to be considered in the refusal of these joints to close completely. Undoubtedly the permanent opening of these joints will eventually become stable when compressive pressures become great enough to overcome those factors tending to keep the joint open.

3. That the rubber asphalt joint seal has not performed in the shorter slab lengths as satisfactorily as expected. This might be attributed in part to the fact that the actual joint opening is greater than was originally estimated.

In considering the design of the test section, it was estimated the seasonal joint opening would be in the vicinity of 0.12 to 0.15 in. for a temperature differential of 80 F. Actual measurements show the average total opening per joint to be 0.17 in. However, this includes a permanent opening of 0.06 in., which really leaves 0.11 in. of actual movement per joint. There have been individual cases where the total joint opening was found to be as much as 0.40 in. and the seal condition was obviously unsatisfactory in these instances. Where the joint openings have been at or near the average of 0.17 in., the seal, in some instances, was in good condition whereas in others it has been in poor condition. Observations were made of the joint seal condition at temperatures between 10 F and 25 F. Photographs were taken of the joints at this time and again in July when temperatures were between 90 F and 100 F. At the latter time, the joint seal condition appeared quite good in all joints due to combination of warm weather and the kneading action of traffic.

4. In general, joint faulting has been slight. The average faulting of the group of joints containing the Type F assembly is comparatively high, but as yet no conclusions have been reached as to the effectiveness of these dowels in the transfer of loads. In November, 1956, the road roughness of the test section was measured with a duplicate model of the BPR Road Roughness Indicator. A roughness index of 114 in. per mile was obtained in the outer wheel track of the travel lane. Unfortunately, no earlier readings are available for this test section alone. However, in 1954, the roughness index obtained on four miles of this roadway which included the test section was 103 in. per mile. While the joint faulting as measured appears slight, the road roughness oscillograph recorder does show a definite roughness at the joints. More data are required to determine the effectiveness of the various dowels insofar as faulting is concerned.

5. That end movement of the 1,800 ft test section during periods of high temperature has been considerably restrained with the result that compressive stresses of 400 to 600 psi may be expected. Undoubtedly, the fact that wood filler was used in the six expansion joints immediately preceding and following the test section accounts for a good deal of the lack of movement at the ends of the test section.

6. That rusting of the dowels at the joint openings has become quite pronounced after three years and is undoubtedly a large factor in the non-uniformity of joint opening.

7. To date, there has been no sign of any physical distress in the pavement of the test section.

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